

## Chapter 3

### Deformation Measurement and Alignment Instrumentation

#### 3-1. General

This chapter describes the different techniques and equipment that are used in measuring external structural deformations.

*a. Geodetic and geotechnical measurements.* The measuring techniques and instrumentation for deformation monitoring have traditionally been categorized into two groups according to the disciplines of professionals who use the techniques:

- geodetic surveys, which include conventional (terrestrial), photogrammetric, satellite, and some special techniques (interferometry, hydrostatic leveling, alignment, etc.)
- geotechnical/structural measurements of local deformations using lasers, tiltmeters, strainmeters, extensometers, joint-meters, plumb lines, micrometers, etc.

*b. Comparison of measurement methods.* Each measurement type has its own advantages and drawbacks. Geodetic surveys, through a network of points interconnected by angle and/or distance measurements, usually supply a sufficient redundancy of observations for the statistical evaluation of their quality and for a detection of errors. They give global information on the behavior of the deformable structure while the geotechnical measurements give very localized and, very frequently, locally disturbed information without any check unless compared with some other independent measurements. On the other hand, geotechnical instruments are easier to adapt for automatic and continuous monitoring than conventional geodetic instruments. Conventional terrestrial surveys are labor intensive and require skillful observers, while geotechnical instruments, once installed, require only infrequent checks on their performance. Geodetic surveys have traditionally been used mainly for determining the absolute displacements of selected points on the surface of the object with respect to some reference points that are assumed to be stable. Geotechnical measurements have mainly been used for relative deformation measurements within the deformable object and its surroundings. However, with the technological progress of the last few years, the differences between the two techniques and their main applications are not as obvious as twenty years ago.

(1) For example, inverted plumb-lines and borehole extensometers, if anchored deeply enough in bedrock below the deformation zone, may serve the same way as, or even better than, geodetic surveys for determining the absolute displacements of the object points. Geodetic surveys with optical and electro-magnetic instruments (including satellite techniques) are always contaminated by atmospheric (tropospheric and ionospheric) refraction, which limits their positioning accuracy to about  $\pm 1$  ppm to  $\pm 2$  ppm (at the standard deviation level) of the distance. So, for instance, given a 500 m average distance between the object and reference points, the absolute displacements of the object points cannot be determined to an accuracy better than about  $\pm 2$  mm at the 95% probability level. In some cases this accuracy is not adequate. On the other hand, precision electro-optical geodetic instruments for electronic distance measurements (EDM) with their accuracies of  $\pm 0.3$  mm over short distances may serve as extensometers in relative deformation surveys. Similarly, geodetic leveling, with an achievable accuracy of better than  $\pm 0.1$  mm over distances of 20 m may provide better accuracy for the tilt determination (equivalent to  $\pm 1$  second of arc) than any local measurements with electronic tiltmeters. Measurements of small concrete cracks can be made to a high degree of accuracy using micrometers--see Chapter 7 for micrometer observation procedures. New developments in three-dimensional coordinating systems with electronic theodolites may provide relative positioning in almost real-time to an accuracy of  $\pm 0.05$  mm

over distances of several meters. The same applies to new developments in photogrammetric measurements with the solid state cameras (CCD sensors). The satellite-based Global Positioning System (GPS), which, if properly handled, offers a few millimeters accuracy in differential positioning over several kilometers. GPS is replacing conventional terrestrial surveys in many deformation studies and, particularly, in establishing the reference networks.

(2) From the point of view of the achievable instrumental accuracy, the distinction between geodetic and geotechnical techniques no longer applies. With the recent technological developments in both geodetic and geotechnical instrumentation, at a cost one may achieve almost any practically needed instrumental resolution and precision, full automation, and virtually real-time data processing. Thus, the array of different types of instruments available for deformation studies has significantly broadened within the last few years. This creates a new challenge for the designers of the monitoring surveys: what instruments to choose, where to locate them, and how to combine them into one integrated monitoring scheme in which the geodetic and geotechnical/structural measurements would optimally complement each other.

### 3-2. Angle and Distance Measurements

Manually operated transits and theodolites have been traditionally used for angle measurement in structural deformation surveying. Distances were measured using precise surveying chains (tapes) or manually operated electronic distance measurement (EDM) devices. Electronic total station devices, such as those shown in Figure 3-1, have largely replaced these older instruments and techniques.

*a. Electronic theodolites.* Over the last two decades, the technological progress in angle measurements has been mainly in the automation of the readout systems of the horizontal and vertical circles of the theodolites. The optical readout systems have been replaced by various, mainly photo-electronic, scanning systems of coded circles with an automatic digital display and transfer of the readout to electronic data collectors or computers. Either decimal units (gons) or traditional sexagesimal units of degrees, minutes, and seconds of arc may be selected for the readout ( $360 \text{ deg} = 400 \text{ gons}$ ). The sexagesimal system of angular units is commonly accepted in North America. As far as accuracy is concerned, electronic theodolites have not brought any drastic improvements in comparison with precision optical theodolites. Some of the precision electronic theodolites, such as the Kern E2 (discontinued production), Leica (Wild) T2002 and T3000, and a few others, are equipped with microprocessor controlled biaxial sensors (electronic tiltmeters) which can sense the inclination (misleveling) of the theodolite to an accuracy better than 0.5 inch and automatically correct not only vertical but also horizontal direction readouts. In optical theodolites in which the inclination is controlled only by a spirit level, errors of several seconds of arc in horizontal directions could be produced when observing along steeply inclined lines of sight. Therefore, when selecting an electronic theodolite for precision surveys, one should always choose one with the biaxial leveling compensator. Atmospheric refraction is a particular danger to any optical measurements, particularly where the line-of sight lies close to obstructions. The gradient of air temperature in the direction perpendicular to the line of sight is the main parameter of refraction.

*b. Three-dimensional coordinating systems.* Two or more electronic theodolites linked to a microcomputer create a three-dimensional (3D) coordinating (positioning) system with real-time calculations of the coordinates. The systems are used for the highest precision positioning and deformation monitoring surveys over small areas. Leica (Wild) TMS and UPM400 (Geotronics, Sweden) are examples of such systems. If standard deviations of simultaneously measured horizontal and vertical angles do not exceed 1 inch, then positions (x, y, z) of targets at distances up to ten meters away may be determined with the standard deviations smaller than 0.05 millimeters. Usually short invar rods of known length are included in the measuring scheme to provide scale for the calculation of coordinates.



**Figure 3-1. Leica TCA 2003 and TM 5100 electronic total stations used for high precision machine alignment and deformation measurements. Accuracy specified at 1.0 mm (distance) and 0.5 sec (angular)**

*c. Electronic Distance Measurements (EDM).* Short range (several kilometers), electro-optical EDM instruments with visible or near infrared continuous radiation are used widely in engineering surveys. The accuracy (standard deviation) of EDM instruments may be expressed in a general form as:

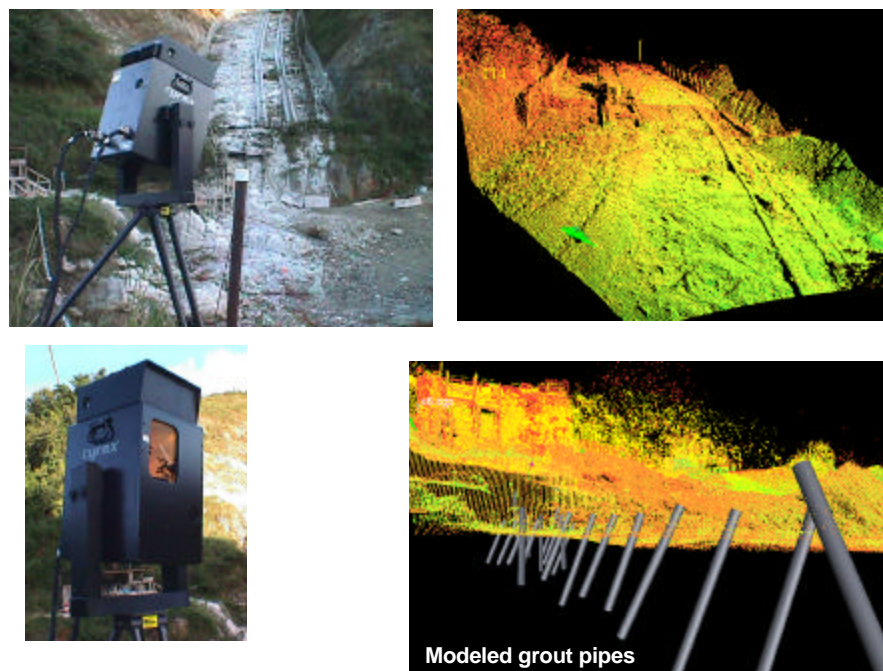
$$\sigma = \text{sqrt} ( a^2 + b^2 \cdot S^2 ) \quad (\text{Eq 3-1})$$

where "a" contains errors of the phase measurement and calibration errors of the so-called zero correction (additive constant of the instrument and of the reflector), while the value of "b" represents a scale error due to the aforementioned uncertainties in the determination of the refractive index and errors in the calibration of the modulation frequency. Typically, the value of "a" ranges from 3 mm to 5 mm. In the highest precision EDM instruments, such as the Kern ME5000, Geomensor CR234 (Com-Rad, U.K.), and Tellurometer MA200 (Tellumat, U.K.), the "a" value is 0.2 mm to 0.5 mm based on a high modulation frequency and high resolution of the phase measurements in those instruments. One recently developed engineering survey instrument is Leica (Wild) DI2002 that offers a standard deviation of 1 mm over short distances. Over distances longer than a few hundred meters, however, the prevailing error in all EDM instruments is due to the difficulty in determining the refractive index.

*d. Pulse type measurement.* Recently, a few models of EDM instruments with a short pulse transmission and direct measurement of the propagation time have become available. These instruments, having a high energy transmitted signal, may be used without reflectors to measure short distances (up to

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200 m) directly to walls or natural flat surfaces with an accuracy of about 10 millimeters. Examples are the Pulsar 500 (Fennel, Germany) and the Leica (Wild) DIOR 3002. Cyra Technologies, Inc has developed automated laser scanning instruments which can be used to scan accurate ( $\pm 5$  mm), real-time detailed models of structures and construction sites--see Figure 3-2.



**Figure 3-2. Real-time laser modeling during 1999 construction of Portugues Dam near Ponce, Puerto Rico--Cyrax Model 2400 (Jacksonville District, Arc Surveying & Mapping, Inc, Cyra Technologies, Inc.)**

*e. Dual frequency instruments.* Only a few units of a dual frequency instrument (Terrameter LDM2 by Terra Technology) are available around the world. They are bulky and capricious in use but one may achieve with them a standard deviation of  $\pm 0.1$  mm  $\pm 0.1$  ppm. Due to a small demand, its production has been discontinued. Research in the development of new dual frequency instruments is in progress.

*f. Total stations.* Any electronic theodolite linked to an EDM instrument and to a computer creates a total surveying station which allows for a simultaneous measurement of the three basic positioning parameters, distance, horizontal direction, and vertical angle, from which relative horizontal and vertical positions of the observed points can be determined directly in the field. Several manufacturers of survey equipment produce integrated total stations in which the EDM and electronic angle measurement systems are incorporated into one compact instrument with common pointing optics, as illustrated in Figure 3-1. Different models of total stations vary in accuracy, range, sophistication of the automatic data collection, and possibilities for on-line data processing. One total station model specifically designed for precision engineering surveys is the Leica (Wild) TC2002 which combines the precision of the aforementioned electronic theodolite, Leica (Wild) T2002, with the precision EDM instrument, Leica (Wild) DI2002, into one instrument with a coaxial optics for both the angle and distance measurements.

g. *Theomat Coaxial Automated Total Station (TCA)*. Leica Company Inc. produces the TCA2003 automated total station instrument, which is designed for conducting deformation monitoring surveys. The TCA2003 system uses a standard tribrach mounting system and internal NiCad batteries or an external 12-volt battery and/or AC power inverter. The user controls measurement functions with a keyboard display. Data collection is carried out via PCMCIA type 1 S-RAM data collector cards having a 2-4 MB capacity (approximately 8000 measurements), that can be directly downloaded to a PC equipped with the proper communications port drivers. It is equipped with a manual-use and/or automatic correcting biaxial compensator to minimize leveling error. The instrument telescope has a 32× lens magnification, so pointing errors are limited to not less than approximately 1 arc-second. The 42-mm objective lens is non-panfocal so that the magnification in this system is fixed. Higher magnification would be desirable for monitoring applications. The angle measurement system uses an absolute encoder with four independent circle readings made at each pointing. An eccentricity in the reading scale any more than 0.5 arc-seconds would be calibrated out at the factory by programming a look-up table of corrections for all possible circle readings. Internally, the EDM tracks a decade modulated infrared (IR) carrier wave having a 0.6 mm resolution at 120 meters. The EDM system specifications are for 1 mm and 1 ppm precision to a single prism at a range of 2500 feet

h. *Automatic Target Recognition (ATR)*. Early automated vision systems were installed in precision theodolites by the 1980's. Its operating components consisted of an external video camera imaging system and a separate servomotor drive. Modern systems are more sophisticated being packaged internally and having an active beam sensing capability. An emitted IR signal is transmitted to the prism that passively reflects the signal back to the instrument. The return spot is imaged on a high-resolution (500 x 500) pixel CCD array. The center of gravity (centroid) is located in relation to the current position of the optical cross-hairs (reticule). An initial calibration process is carried out immediately after setting up the instrument, where a reference object is sighted so that the fixed orientation of the telescope is registered to the ATR image coordinates. To run the system after calibration, a series of targets are sighted so the instrument can be trained to their location at least once. With the approximate coordinates of each target prism stored in memory, the ATR system can then take over the pointing, reading, and measuring functions completely within the instrument. Target search radius, data rejection thresholds, and other controls can be programmed into the operating menus by the user. The search pattern is set by default to one-third of the telescope field of view, but this range can be narrowed to provide better search and recognition performance once the instrument has been trained to a given point. Factory reliability tests on the servomotor drive have proven continuous operation of the system over four consecutive years in a continuous measurement mode.

i. *Data communications and software*. Recorded data can be downloaded to an external file or automatically communicated via RS-232, UHF radio link, spread spectrum radio, or radio modem over up to one kilometer. Although data is transmitted in relatively short streams, its onboard communications capability is not yet Internet (TCICP) compatible. Software applications for the system range from writing ASCII file output to pre-packaged analysis software (APS Win) for tracking and monitoring changes in the measurements. Two versions of the APS Win Software can be purchased. The first is a full system that has data collection, processing, and analysis capabilities. The second is a light version with only data logging features. Custom data collector software can also be programmed manually using a software package known as Geobasic, which provides a high-level programmers development environment. The APS Win package can be used to remotely configure measurement sequences, such as; the number of targets, target sighting sequence, and time interval between measurements. However, the repetition time can be set to no less than once a minute. Downloading the data through a PC computer is "drag & drop" via a survey office (file manager) software package. A proprietary data format is used to collect the data, which is translated, into readable ASCII text files by the GSI editor program.

*j. Survey robots.* For continuous or frequent deformation measurements, a fully automatic system based on computerized and motorized total stations has recently been developed. The first commercial system was Georobot. Recent advanced systems include for example, the Geodimeter 140 SMS (Slope Monitoring System) and the Leica (Wild) APS and Georobot III systems based on the motorized TM 3000 series of Leica (Wild) electronic theodolites linked together with any Leica (Wild) DI series of EDM. These can be programmed for sequential self-pointing to a set of prism targets at predetermined time intervals, can measure distances and horizontal and vertical angles, and can transmit the data to the office computer via a telemetry link. Similar systems are being developed by other manufacturers of surveying equipment. The robotic systems have found many applications, particularly in monitoring high walls in open pit mining and in slope stability studies. Generally, the accuracy of direction measurements with the self-pointing computerized theodolites is worse than the measurements with manual pointing.

### 3-3. Differential Leveling

*a. General.* Differential leveling provides height difference measurements between a series of benchmarks. Vertical positions are determined to very high accuracy ( $\pm 1$  mm) over short distances (10-100's of meters) using precision levels. Two major classes of precision levels commonly used for making deformation measurements are automatic levels and digital levels.

*b. Automatic levels.* The old method of geometrical leveling with horizontal lines of sight (using spirit or compensated levels) is still the most reliable and accurate, though slow, surveying method. With high magnification leveling instruments, equipped with the parallel glass plate micrometer and with invar graduated rods, a standard deviation smaller than 0.1 mm per set-up may be achieved in height difference determination as long as the balanced lines of sight do not exceed 20 meters. In leveling over long distances (with a number of instrument set-ups) with the lines of sight not exceeding 30 m, a standard deviation of 1 mm per kilometer may be achieved in flat terrain. The influence of atmospheric refraction and earth curvature is minimized by balancing the lines of sight between the forward and backward leveling rods. A dangerous accumulation of refraction error, up to 15 mm for each 100 m difference in elevation, may take place along moderately inclined long routes due to unequal heights of the forward and backward horizontal lines above the terrain.

*c. Digital levels.* The recently developed Leica NA2000 and NA3000 digital automatic leveling systems with height and distance readout from encoded leveling rods (Figure 3-3) has considerably increased the speed of leveling (by about 30%) and decreased the number of personnel needed on the survey crew. Some users of the digital level complain that its compensating system demonstrates systematic deviations in windy weather and, therefore, cannot be classified as a high precision level unless some improvements are introduced by the manufacturer.

*d. Tilt measurements by leveling.* Monolith tilt on dams can be determined from leveling observations using the dimensions and tilting axis of the object. For example, a well-spaced (e.g., rectangular) four-point configuration of points provides the attitude of a plane that can be solved by least squares surface fitting to the measurements. The required survey data inputs are the height differences between the points and an absolute height tied to at least one point, either transferred or assumed from a given reference. The unknown two-axis tilt parameters ( $\alpha_x$  and  $\alpha_y$ ) are derived from the solution of the equation for the plane.



Figure 3-3. Lieca NA 2002 automated digital level and section from bar-coded invar level rod

### 3-4. Total Station Trigonometric Elevations

*a. Zenith angle methods.* High precision electronic theodolites and EDM equipment allow for the replacement of geodetic leveling with more economical trigonometric height measurements. An accuracy better than 1 mm may be achieved in height difference determination between two targets 200 m apart using precision electronic theodolites for vertical angle measurements and an EDM instrument. The measurements must be performed either reciprocally, with two theodolites simultaneously, or from an auxiliary station with equal distances to the two targets (similar methodology as in spirit leveling) to minimize atmospheric refraction effects.

*b. Measurement accuracy.* Zenith angle heighting accuracy is practically independent of the height differences and is especially more economical than conventional leveling in hilly terrain, and in all situations where large height differences between survey stations are involved. Trigonometric height traversing (reciprocal or with balanced lines of sight) with precision theodolites and with the lines of sight not exceeding 250 m can give a standard deviation smaller than 2 mm per kilometer. For standard height transfer applications, with automatic data collection and on-line processing, measurements are achieved independent of the terrain configuration. The refraction error is still the major problem with increasing the accuracy of trigonometric leveling.

### 3-5. Global Positioning System (GPS)

*a. General.* The satellite Global Positioning System (shown in Figures 3-4 and 3-5) offers advantages over conventional terrestrial methods. Intervisibility between stations is not strictly necessary, allowing greater flexibility in the selection of station locations than for terrestrial geodetic surveys. Measurements can be taken during night or day, under varying weather conditions, which makes GPS measurements economical, especially when multiple receivers can be deployed on the structure during the survey. With the recent developed rapid static positioning techniques, the time for the measurements at

each station is reduced to a few minutes. Reference EM 1110-1-1003, NAVSTAR Global Positioning System Surveying.



**Figure 3-4. GPS equipment setup on a concrete hydropower dam - spillway and intake structure**

*b. Measurement accuracy.* GPS is still a new and not perfectly known technology from the point of view of its optimal use for deformation surveying and understanding related sources of error. The accuracy of GPS relative positioning depends on the distribution (positional geometry) of the observed satellites and on the quality of the observations. Several major sources of error contaminating the GPS measurements are:

- signal propagation errors--tropospheric and ionospheric refraction, and signal multipath,
- receiver related errors--antenna phase center variation, and receiver system noise,
- satellite related errors--such as orbit errors and bias in the fixed station coordinates.

GPS errors relative to deformation survey applications are discussed in detail in Chapter 8.



**Figure 3-5. Standard GPS equipment for precise surveying. From left to right; graduated rods for antenna height measurement, GPS antenna with ground plane, tribrach, antenna/tribrach adapter, antenna cable, data download cable, surveyors tripod, GPS receiver, camcorder batteries, power cord for support module, 12V battery with attached cable, support module for data downloading.**

*c. GPS positioning accuracy.* Experience with the use of GPS in various deformation studies indicate that with the available technology the accuracy of GPS relative positioning over areas of up to 50 km in diameter can be expressed in terms of the variance of the horizontal components of the GPS baselines over a distance (S):

$$\sigma^2 = (3\text{mm})^2 + (10^{-6} \cdot S)^2 \quad (\text{Eq 3-2})$$

Systematic biases (rotations and change in scale of the network) are identified and eliminated through proper modeling at the stage of the deformation interpretation. The accuracy of vertical components of the baselines is 1.5 to 2.5 times worse than the horizontal components. Systematic measurement errors over short distances (up to a few hundred meters) are usually negligible and the horizontal components of the GPS baselines can be determined with a standard deviation of 3 mm or even smaller. Recent improvements to the software for the GPS data processing allow for an almost real time determination of changes in the positions of GPS stations.

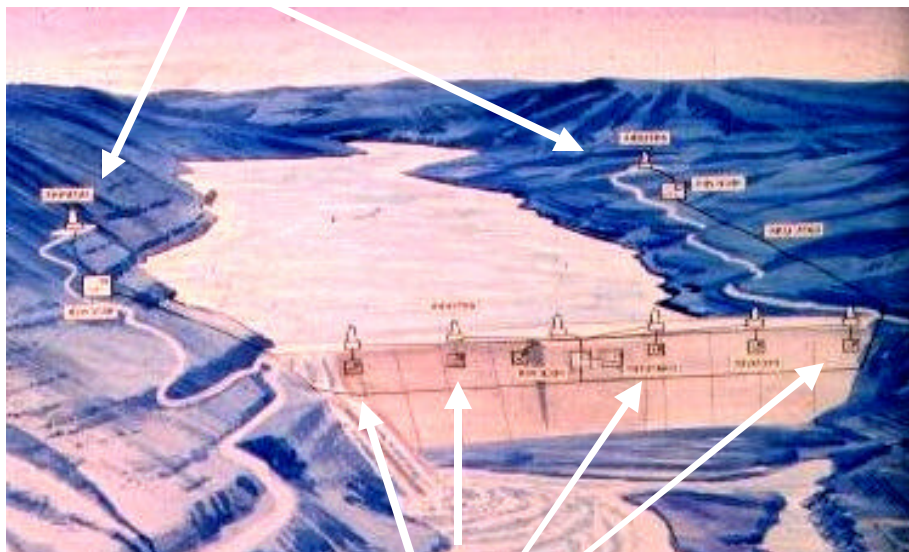
*d. Systematic GPS errors.* Different types of errors affect GPS relative positioning in different ways. Some of the errors may have a systematic effect on the measured baselines producing scale errors and rotations. Due to the changeable geometrical distribution of the satellites and the resulting changeable systematic effects of the observation errors, repeated GPS surveys for the purpose of monitoring deformations can affect derived deformation parameters (up to a few ppm). Attention to the systematic influences should be made when a GPS network is established along the shore of a large body of water and measurements are performed in a hot and humid climate. The solution for systematic parameters in a GPS network may be obtained by:

- combining GPS surveys of some baselines (with different orientation) with terrestrial surveys of a compatible or better accuracy,
- establishing several points outside the deformable area (fiducial stations) which would serve as reference points.

These aspects must be considered when designing GPS networks for any engineering project.

*e. Automated GPS surveys.* USACE developed a fully automated system for high-precision deformation surveys with GPS. With the Continuous Deformation Monitoring System (CDMS) GPS antennas are located at multiple points on the structure. At least two other GPS antennas must be located over reference points that are considered stable. The GPS antennas are connected to computers using a data telemetry link. A prototype CDMS system used 10-channel Trimble 4000SL and Trimvec post processing software. An operator could access the on-site computer network through a remote hook-up in the office. In 1989 the system was installed at the Dworshak Dam on the Clearwater River near Orofino, Idaho (Figure 3-6). The demonstration results show that CDMS can give accuracies of 3 mm both horizontally and vertically over a 300 m baseline.

#### GPS reference stations



#### Target points on dam

Figure 3-6. 1989 Concept sketch depicting GPS deformation monitoring surveys on a dam. GPS monitoring was first applied at Dworshak Dam, Idaho. (Walla Walla District)

Although GPS does not require the intervisibility between the observing stations it requires an unobstructed view to the satellites which limits the use of GPS only to reasonably open areas. One should also remember that there might be some additional sources of errors (e.g., multipath, etc.) in GPS measurements.

*f. GPS receiver specifications.* When performing GPS based deformation surveys, the receiver used must be geodetic quality, multi-channel, single frequency, and capable of one second data sampling. The receiver should also be capable of recording the GPS carrier frequency, receiver clock time, and signal strength for each data sample. A GPS receiver is required for each reference station in the reference network. The same model receiver/antenna combination should be used for each setup. Pre-processing of GPS survey data, at a minimum, must include determination of the 3D coordinate differences and associated variance-covariance matrix in the 3D coordinate system for all baselines observed, and data screening to eliminate possible outliers. When performing GPS-based deformation surveys, procedures should be done in accordance with the guidance in Chapter 8 of this manual.

### 3-6. Photogrammetric Techniques

*a. General.* If an object is photographed from two or more survey points of known relative positions (known coordinates) with a known relative orientation of the camera(s), relative positions of any identifiable object points can be determined from the geometrical relationship between the intersecting optical rays which connect the image and object points. If the relative positions and orientation of the camera are unknown, some control points on the object must be first positioned using other surveying techniques. Aerial photogrammetry has been extensively used in determining ground movements in ground subsidence studies in mining areas, and terrestrial photogrammetry has been used in monitoring of engineering structures. The main advantages of using photogrammetry are the reduced time of field work; simultaneous three dimensional coordinates; and in principle an unlimited number of points can be monitored. The accuracy of photogrammetric point position determination has been much improved in the past decade, which makes it attractive for high precision deformation measurements.

*b. Terrestrial photogrammetry.* Special cameras with minimized optical and film distortions must be used in precision photogrammetry. Cameras combined with theodolites (phototheodolites), for instance the Wild P-30 model, or stereocameras (two cameras mounted on a bar of known length) have found many applications in terrestrial engineering surveys including mapping and volume determination of underground excavations and profiling of tunnels. The accuracy of photogrammetric positioning with special cameras depends mainly on the accuracy of the determination of the image coordinates and the scale of the photographs. The image coordinates may, typically, be determined with an accuracy of about 10  $\mu\text{m}$ , though 3  $\mu\text{m}$  is achievable. The photo scale may be approximately expressed as:

$$\text{Photo Scale} = f / S \quad (\text{Eq 3-3})$$

where

$f$  = the focal length of the camera lens

$S$  = the distance of the camera from the object.

Using a camera with  $f = 100 \text{ mm}$ , at a distance  $S = 100 \text{ m}$ , with the accuracy of the image coordinates of 10  $\mu\text{m}$ , the coordinates of the object points can be determined with the accuracy of 10 mm. Special large format cameras with long focal length are used in close range industrial applications of high precision. For instance, the model CRC-1 (Geodetic Services, Inc.) camera with  $f = 240 \text{ mm}$ , can give sub-millimeter accuracy in 'mapping' objects up to a few tens of meters away. Recently, solid state cameras with CCD (charge couple device) sensors have become available for close range

photogrammetry in static as well as in dynamic applications. Continuous monitoring with real time photogrammetry becomes possible with the new developments in CCD cameras and digital image processing techniques.

*c. Photogrammetric standards.* When performing photogrammetric based deformation surveys, only metric cameras will be used. Typically, only one camera is necessary as it is moved from station to station. The instrument used for image coordinate measurement (e.g., monocomparator, stereocomparator, or analytical stereocomparator) will be capable of 1 micron or better resolution.

*d. Photogrammetry operations.* When performing photogrammetric based deformation surveys, the metric camera used will be mounted in or on a suitable camera platform (e.g., camera tripod). During exposure, movement of the camera will be minimized. If using an airplane or helicopter for the platform, a camera with an image motion compensator must be used. Typically, 5 to 20 exposure stations are necessary to insure sufficient precision for the object point coordinates are determined. To ensure the whole photo taking portion of the survey is performed correctly, it is highly recommended that only experienced personnel be used for this phase of the survey. The photogrammetric reduction process also should be done by experienced personnel trained in image coordinate measurement with the appropriate equipment. If practicable, it is recommended that this process be automated in order to eliminate potential gross errors possible with self-calibration. EM 1110-1-1000, Photogrammetric Mapping, and photogrammetric product manufacturer guidelines should be referred to for more specifics on the photogrammetric process.

*e. Pre-processing photo control survey data.* Pre-processing of conventional survey data consists of applying statistical tests at the time the observations are made in order to reject probable outliers, and applying atmospheric, instrument calibration, standardization, and geometric corrections so data can be imported to subsequent network adjustment software. Pre-processing of conventional survey observations can either be done manually or by appropriate verified and validated PC based programs.

*f. Pre-processing photogrammetric survey data.* Pre-processing of photogrammetric based survey data will include the screening of measured image coordinates in order to reject observation which are outliers and determination of 3D object coordinates and associated variance-covariance matrix in the local coordinate system. Determination of the 3D object coordinates should be accomplished by a computer based bundle adjustment program with self-calibration. Also, in the bundle adjustment, the focal length, position of the principal point, coefficients of radial and asymmetric lens distortion, and photographic media unflatness will be treated as weighted unknowns. Atmospheric refraction can be neglected if the exposure distance is kept to what is recommended.

### 3-7. Alignment Measurements

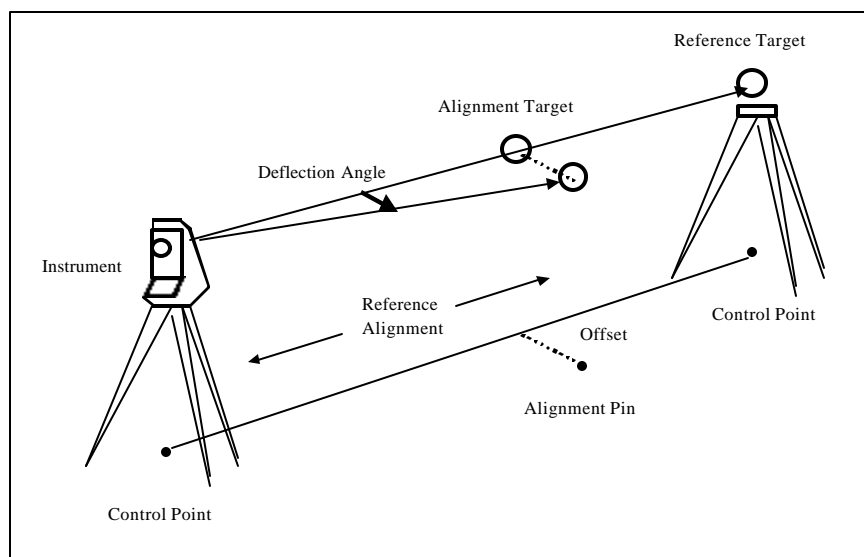
*a. General.* Alignment surveys cover an extremely wide spectrum of engineering applications from the tooling industry, through measurements of amplitude of vibrations of engineering structures, to deformation monitoring of nuclear accelerometers several kilometers long. Each application may require different specialized equipment. The methods used in practice may be classified according to the method of establishing the reference line:

- mechanical method in which stretched wire (e.g., steel, nylon) establishes the reference line,
- direct optical method (collimation) with optical line of sight or a laser beam to mark the line,
- diffraction method where a reference line is created by projecting a pattern of diffraction slits.

*b. Mechanical methods.* Mechanical alignment methods with tensioned wires used as the reference lines have found many applications, including dam deformation surveys. This is due to their

simplicity, high accuracy, and easy adaptation to continuous monitoring of structural deformations using inductive sensors applicable over distances up to a few hundred meters. Accuracies of 0.1 mm are achievable using mechanical alignment methods.

*c. Direct optical methods.* Direct optical methods (Figure 3-7) utilize either an optical telescope and movable targets with micrometric sliding devices or a collimated laser beam (projected through the telescope) and movable photo-centering targets. Besides the aforementioned influence of atmospheric refraction, pointing and focusing are the main sources of error when using optical telescopes. Refer to Chapter 7 for details on performing micrometer alignment observations.



**Figure 3-7. Direct optical alignment technique. Deflection angle method used to measure baseline offsets in conventional alignment surveys**

*d. Aligning telescopes.* Special aligning telescopes with large magnification (up to 100×) are available from, among others, Fennel-Cassell (Germany) and Zeiss-Jena (Germany). Aligning telescopes for the tooling industry and machinery alignment are available in North America from Cubic Precision. When the optical line of sight is replaced by a collimated laser beam, then the accuracy of pointing may be considerably improved if special self-centering laser detectors with a time integration of the laser beam energy are used. The use of laser allows for automation of the alignment procedure and for continuous data acquisition. Attention must be paid to the stability of the laser cavity when using the laser beam directly as the reference line. A directional drift of the laser beam as high as 4 inches per deg C may occur due to thermal effects on the laser cavity. This effect is decreased by a factor of the magnification when projecting the laser through a telescope.

*e. Diffraction methods.* In diffraction alignment methods, a pinhole source of monochromatic (laser) light, the center of a plate with diffraction slits, and the center of an optical or photoelectric sensor are the three basic points of the alignment line. If two of the three points are fixed in their position, then the third may be aligned by centering the reticule on the interference pattern created by the diffraction grating. It should be pointed out that movements of the laser and of its output do not influence the accuracy of this method of alignment because the laser serves only as a source of monochromatic light placed behind the pinhole and not as the reference line. Therefore, any kind of laser may be employed in this method, even the simplest and least expensive ones, as long as the output power requirements are

satisfied. Various patterns of diffraction slits are used in practice. The highest accuracy and the longest range are obtained with the so-called Fresnel zone plates that act as focusing lenses. For instance, rectangular Fresnel zone plates with an electro-optical centering device were used in alignment and deformation measurements of a 3 km long nuclear accelerator giving relative accuracy (in a vacuum) of  $10^{-7}$  of the distance. In the open atmosphere, the thermal turbulence of air seems to have a smaller effect when using the Fresnel zone plates than in the case of direct optical alignment. The laser diffraction alignment methods have successfully been applied in monitoring both straight and curved (arch) dams using self-centering targets with automatic data recording.

*f. Micrometer translation stages.* Developments in the manufacture of translation stages for scientific and laboratory use (such as for laser and optical alignment work), as well as other specialized products used in the field of industrial metrology include a broad array of alignment measurement tools (such as scales, precise micrometers, and right angle prisms). Modern linear translation stages can reliably provide extremely high resolution (1/1000 inch at one-sigma) with very stable material and mechanical properties. Translation stages with large travel ranges are available to adapt these off-the-shelf devices to monitoring applications, especially alignment surveys.

### 3-8. Extension and Strain Measurements

*a. Types of extensometers.* Various types of instruments, mainly mechanical and electro-mechanical, are used to measure changes in distance in order to determine compaction or upheaval of soil, convergence of walls in engineering structures and underground excavations, strain in rocks and in man-made materials, separation between rock layers around driven tunnels, slope stability, and movements of structures with respect to the foundation rocks. Depending on its particular application, the same instrument may be named an extensometer, strainmeter, convergencemeter, or fissuremeter. The various instruments differ from each other by the method of linking together the points between which the change in the distance is to be determined and the kind of sensor employed to measure the change. The links in most instruments are mechanical, such as wires, rods, or tubes. The sensors usually are mechanical, such as calipers or dial gauges. In order to adapt them to automatic and continuous data recording, electric transducers can be employed using, for instance, linear potentiometers, differential transformers, and self-inductance resonant circuits. In general, when choosing the kind of transducer for automatic data acquisition, one should consult with an electronics specialist on which kind would best suit the purpose of the measurements in the given environmental conditions. One should point out that the precision EDM instruments, as described earlier with their accuracy of 0.3 mm over short distances, may also be used as extensometers particularly when the distances involved are several tens of meters long. If an extensometer is installed in the material with a homogeneous strain field, then the measured change (  $dl$  ) of the distance (  $l$  ) gives directly the strain component (  $e$  ):

$$e = dl/l \quad \text{(Eq 3-4)}$$

in the direction of the measurements. To determine the total strain tensor in a plane (two normal strains and one shearing), a minimum of three extensometers must be installed in three different directions.



Various slide and vernier calipers

Inside & outside micrometers



Figure 3-8. Assortment of Starrett micrometers and calipers that can be used for measuring short distances in concrete structures to an accuracy of 0.0005 inch or better

*b. Wire and tape extensometers.* Maintaining a constant tension throughout the use of the wire or tape extensometer is very important. In some portable extensometers, the constant tensioning weight has been replaced by precision tensioning springs. One should be careful because there are several models of spring tensioned extensometers on the market which do not provide any means of tension calibration. As the spring ages, these instruments may indicate false expansion results unless they are carefully calibrated on a baseline of constant length before and after each measuring campaign.

(1) Invar wire strain gauge. Among the most precise wire extensometers are the Kern Distometer (discontinued production) and the CERN Distinvar (Switzerland). Both instruments use invar wires and special constant tensioning devices which, if properly calibrated and used, can give accuracies of 0.05 mm or better in measurements of changes of distances over lengths from about 1 m to about 20 meters. Invar is a capricious alloy and must be handled very carefully to avoid sudden changes in the length of the wire. When only small changes in temperature are expected or a smaller precision (0.1 mm to 1 mm) is required, then steel wires or steel tapes are more comfortable to use.

(2) Vibrating wire strain gauge. Special high precision strainmeters of a short length (up to a few decimeters) are available for strain measurements in structural material and in homogeneous rocks. An example is a vibrating wire strain gauge available from Rocktest (Irad Gage). The instrument employs a 150 mm steel wire in which the changeable resonant frequency is measured. An accuracy of one microstrain ( $10^{-6}$ ) is claimed in the strain measurements which corresponds to 0.15  $\mu\text{m}$  relative displacements of points over a distance of 150 mm.

*c. Rod, tube, and torpedo extensometers.* Steel, invar, aluminum, or fiberglass rods of various lengths, together with sensors of their movements, may be used depending on the application. Multiple point measurements in boreholes or in trenches may be made using either a parallel arrangement of rods anchored at different distances from the sensing head, or a string (in series) arrangement with intermediate sensors of the relative movements of the rods. A typical accuracy of 0.1 mm to 0.5 mm may be achieved up to a total length of 200 m (usually in segments of 3 m to 6 m). The actual accuracy depends on the temperature corrections and on the quality of the installation of the extensometer. When installing rods in plastic conduit (usually when installing in boreholes), the friction between the rod and the conduit may significantly distort the extensometer indications if the length of the extensometer exceeds a few tens of meters. The dial indicator readout may be replaced by potentiometric or other transducers with digital readout systems. Telescopic tubes may replace rods in some simple applications, for instance, in measurements of convergence between the roof and floor of openings in underground mining. Several models of torpedo borehole extensometers and sliding micrometers are available from different companies producing geotechnical instrumentation. For example, Extensofor (Telemac, France) consists of a 28 mm diameter torpedo 1.55 m long with an inductance sensor at each end. Reference rings on the casing are spaced within the length of the torpedo. The sensors and reference rings form the inductance oscillating circuits. The torpedo is lowered in the borehole and stopped between the successive rings recording changes in distances between the pairs of rings with a claimed accuracy of 0.1 millimeter. Boreholes up to several hundreds of meters long can be scanned.

*d. Interferometric measurements of linear displacements.* Various kinds of interferometers using lasers as a source of monochromatic radiation are becoming common tools in precision displacement measurements. A linear resolution of 0.01  $\mu\text{m}$ , or even better, is achievable. One has to remember, however, that interferometric distance measurements are affected by atmospheric refractivity in the same way as all EDM systems. Therefore, even if temperature and barometric pressure corrections are applied, the practical accuracy limit is about  $10^{-6}\text{S}$  (equivalent to 1  $\mu\text{m}$  per meter). Thermal turbulence of air limits the range of interferometric measurements in the open atmosphere to about 60 meters. The laser interferometer has found many industrial and laboratory applications in the measurement of small displacements and the calibration of surveying instruments.

*e. Use of optical fiber sensors.* A new development in the measurements of extensions and changes in crack-width employs a fully automatic extensometer that utilizes the principle of electro-optical distance measurements within fiber optic conduits. The change in length of the fiber optic sensors are sensed electro-optically and are computer controlled.

*f. Precise concrete crack measurements.* Distances between cracks in concrete structures are typically measured using precision micrometers or calipers, such as those as shown in Figure 3-8. Details on micrometer crack observing procedures are covered in Chapter 7.

### 3-9. Tilt and Inclination Measurements

*a. Methods of tilt measurement.* The measurement of tilt is usually understood as the determination of a deviation from the horizontal plane, while inclination is interpreted as a deviation from the vertical. The same instrument that measures tilt at a point can be called either a tiltmeter or an inclinometer depending on the interpretation of the results. Geodetic leveling techniques can achieve an accuracy of 0.1 mm over a distance of 20 m, which would be equivalent to about 1.0 inch of angular tilt. This accuracy is more than sufficient in most engineering deformation measurements. Various in-situ instruments are used when higher accuracy or continuous or very frequent collection of data on the tilt changes is necessary:

- Engineering Tiltmeters and Inclinometers
- Suspended and Inverted Plumb Lines
- Hydrostatic Levels

Other specialized instruments such as mercury/laser levels have been developed but are not commonly used in practice.

*b. Tiltmeters and inclinometers.* There are many reasonably priced models of various liquid, electrolytic, vibrating wire, and pendulum type tiltmeters that satisfy most of the needs of engineering surveys. Particularly popular are servo-accelerometer tiltmeters with a small horizontal pendulum. They offer ruggedness, durability, and low temperature operation. The output signal (volts) is proportional to the sine of the angle of tilt. The typical output voltage range for tiltmeters is  $\pm 5$  V, which corresponds to the maximum range of the tilt. Angular resolution depends on the tilt range of the selected model of tiltmeter and the resolution of the voltmeter (e.g., 1 mV). There are many factors affecting the accuracy of tilt sensing. A temperature change produces dimensional changes of the mechanical components, changes in the viscosity of the liquid in the electrolytic tiltmeters, and of the damping oil in the pendulum tiltmeters. Drifts of tilt indications and fluctuations of the readout may also occur. Thorough testing and calibration are required even when accuracy requirements are not very high. Tiltmeters have a wide range of applications. A series of tiltmeters if arranged along a terrain profile may replace geodetic leveling in the determination of ground subsidence. Similarly, deformation profiles of tall structures may be determined by placing a series of tiltmeters at different levels of the structure. A popular application of tiltmeters in geomechanical engineering is in slope stability studies and in monitoring embankment dams using the torpedo (scanning) type borehole inclinometers (usually the servo-accelerometer type tiltmeters). The biaxial inclinometers are used to scan boreholes drilled to the depth of an expected stable strata in the slope. By lowering the inclinometer on a cable with marked intervals and taking readings of the inclinometer at those intervals, a full profile of the borehole and its changes may be determined through repeated surveys. Usually the servo-accelerometer inclinometers are used with various ranges of inclination measurements, for instance,  $\pm 6$  deg,  $\pm 54$  deg, or even  $\pm 90$  deg. If a 40 m deep borehole is measured every 50 cm with an inclinometer of only 100 inch accuracy, then the linear lateral displacement of the collar of the borehole could be determined with an accuracy of 2 millimeters. Fully automatic (computerized) borehole scanning inclinometer systems with a telemetric data acquisition have been designed for monitoring slope stability.

*c. Suspended and inverted plumb lines.* Two kinds of mechanical plumbing are used in controlling the stability of vertical structures:

- (1) Suspended Plumb Lines,
- (2) Floating or Inverted Plumb Lines.

Inverted plumb lines have an advantage over suspended plumb lines in the possibility of monitoring absolute displacements of structures with respect to deeply anchored points in the foundation rocks that may be considered as stable. In the case of power dams, the depth of the anchors must be 50 m or even more below the foundation in order to obtain absolute displacements of the surface points. If invar wire is used for the inverted plumb line, vertical movements of the investigated structure with respect to the bedrock can also be determined. Caution must be used in installing plumb lines. If the plumb line is installed outside the dam, a vertical pipe of a proper inner diameter should be used to protect the wire from the wind. The main concern with floating plumb lines is to ensure verticality of the boreholes so that the wire of the plumb line has freedom of motion. The tank containing the float is generally filled with oil or with water to which some anti-freeze can be added. The volume of the float should be such as to exert sufficient tension on the wire. Thermal convection displacements in a float tank may easily develop from thermal gradients that may affect measurements--requiring the whole tank to be thermally

insulated. Several types of recording devices that measure displacements of structural points with respect to the vertical plumb lines are produced by different companies. The simplest are mechanical or electromechanical micrometers. With these, the plumb wire can be positioned with respect to reference lines of a recording (coordinating) table to an accuracy of  $\pm 0.1$  mm or better. Traveling microscopes may give the same accuracy. Automatic sensing and recording is possible, for instance, with a Telecoordinator (Huggenberger, Switzerland) and with a Telependulum (Telemac, France). Automated vision systems use CCD video cameras to image the plumb line with a resolution of about 3 micrometer over a range of 75 mm. Two sources of error that may sometimes be underestimated by users are the influence of air currents and the spiral shape of wires. The plumb line should be protected within a pipe (e.g., PVC tube) with openings only at the reading tables to reduce the influence of the air pressure.

*d. Optical plummets.* High precision optical plummets (e.g., Leica ZL (zenith) and NL (nadir) plummets) offer accuracy of up to 1/200,000 for precise centering, and both can be equipped with laser. Atmospheric refraction remains as a major source of error for optical instruments.

*e. Hydrostatic leveling.* If two connected containers are partially filled with a liquid, then the heights  $h_1$  and  $h_2$  of the liquid in the containers are related through the hydrostatic equation

$$h_1 + P_1 / (g_1 r_1) = h_2 + P_2 / (g_2 r_2) = \text{constant} \quad (\text{Eq 3-5})$$

where  $P$  is the barometric pressure,  $g$  is gravity, and  $r$  is the density of the liquid which is a function of temperature. The above relationship has been employed in hydrostatic leveling. The ELWAAG 001 (Bayernwerke, Germany) is a fully automatic instrument with a traveling (by means of an electric stepping motor) sensor pin that closes the electric circuit upon touching the surface of the liquid. Hydrostatic leveling is frequently used in the form of a network of permanently installed instruments filled with a liquid and connected by hose-pipes to monitor change in height differences of large structures. The height differences of the liquid levels are automatically recorded. The accuracy ranges from 0.1 mm to 0.01 mm over a few tens of meters depending on the types of instruments. The main factor limiting the survey accuracy is the temperature effect. To reduce this effect the instrument must either be installed in a place with small temperature variations, or the temperature along the pipes must be measured and corrections applied, or a double liquid (e.g., water and mercury) is employed to derive the correction for this effect. Water of a constant temperature is pumped into the system just before taking the readings for the highest accuracy applications. The instruments with direct measurement of the liquid levels are limited in the vertical range by the height of the containers. This problem may be overcome if liquid pressures are measured instead of the changes in elevation of the water levels, where pneumatic pressure cells or pressure transducer cells may be used.

### 3-10. Non-Geodetic Measurements

*a. General.* Deformation of large structures (e.g., dams) is caused mainly by reservoir loads, temperature, self-weight of the dam, and earth pressure. A monitoring system should therefore include regular measurements of the reservoir level and temperature and pressure data.

*b. Reservoir level measurement.* Reservoir levels today should be measured with pressure balances. Double checking the measurements must be done and can be facilitated by installing a manometer on either an existing or new pipe connected to the reservoir. The measurement range should extend at least as far as the dam crest allowing observation and judgment of the flood risk and assessment of peak inflows.

*c. Temperature measurement.* Temperature measurement is required to determine the impact of temperature variations on the structure itself, as well as whether precipitation consists of rain or snow and, if applicable, whether the snow melt period has begun. Temperature measurement should be done at least daily. The thermometers should be placed at various locations within the dam, either embedded in the structure itself or within drillholes. Redundancy should be provided for by using a greater number of thermometers than otherwise would be required.

*d. Precipitation measurement.* Precipitation measurement should be done by using a precipitation gauge. Daily readings are recommended. The gauging station does not need to be located at the dam site, but should not be too far away so as to not be representative of the precipitation level at the structure itself. Every large structure has some form of seepage through the structure itself or its foundation, even with a grout curtain. In concrete dams, seepage typically is small and limited to permeable areas of the concrete, joints, and contact between rock and concrete. Any abnormal seepage is an immediate warning that something may be wrong with the structure or foundation. In general, seepage flows cause uplift pressure which must be monitored in view of its critical impact on the overall stability of the structure. In embankment (i.e., earthen) dams, seepage flow through the structure itself is similar to that observed in its foundation as the material from which both are made are pervious. Seepage flows not only cause uplift pressure in these structures, but also pore-water pressure. The pattern of seepage and water pressures on the structure (especially on the foundation and impervious core) has a significant impact on the behavior of the dam.

*e. Seepage rate.* The total seepage rate is the seepage at the face of the structure taken as a whole. Seepage rate can be measured volumetrically by using a calibrated container and a stopwatch or a gauging weir or flume. Partial seepage rates are taken in isolated zones of the structure found to be representative for the area examined. Such rates should be monitored periodically. In the course of monitoring seepage, if an abnormality (i.e., a change in normal seepage rate) is detected, the critical zone and cause of the seepage is easier to identify.

*f. Chemical property analysis.* If the structure is constructed of soluble or easily erodable material, the seepage should be monitored for turbidity and chemical content also. Doing so will permit the assessment of the overall stability of the embankment and foundation materials.

*g. Pore-water pressure measurement.* Structures usually are designed with specific pore-water pressure values that should not be exceeded. Pressure cells typically are designed or built into the structure themselves to measure pore-water pressure. The linking together of several cells forms a profile for the structure. The greater the number of measurement profiles and number of cells per profile, the more useful the data obtained will be. Even though pressure cells can be installed in structures themselves, rehabilitation of existing ones is not always practicable. Where pressure cells cannot be used to monitor pore-water pressure, the phreatic line in selected points will be monitored. Standpipe piezometers mounted in the embankment at several cross sections should be used to monitor the phreatic line.

*h. Uplift pressure.* Seepage underneath a structure causes uplift pressure that can severely alter the stabilizing effect of the structure's self weight. Uplift pressure can be reasonably controlled by a grout curtain and drainage holes, but uplift pressure and the physical effectiveness of these control measures should be carefully monitored. Piezometers connected to a manometer are a reliable means to measure the uplift pressure in cross-sections and several points on the upstream and downstream face of the structure.

*i. Discharge measurement.* If the foundation is being drained, drainage discharge should be monitored by either volumetric gauging or gauging weir. Any change in flow rate may be indicative of

clogging in the drainage system. If possible, the discharge of any spring, river, stream, or flood control structure downstream of the structure should be monitored.

### 3-11. Optical Tooling Technology

*a. General.* This section discusses modern optical metrology. A set of methods known in industry as optical tooling is used to create precise lines and planes in space from which measurements are made using light.

*b. Definition of optical tooling.* Optical tooling is a means for establishing and utilizing a line of sight (LOS) to obtain precise reference lines and reference planes from which accurate measurements are made with position sensitive targets [Williams, 1989]. Measurements are made by a person interpreting a scale or optical micrometer by looking through an alignment telescope, or the lines and planes are created by a laser with digital measurements. Optical tooling uses the principle that light travels in straight lines so as to enable precise measurements and level lines with every point is perpendicular to the force of gravity (e.g., plumb lines can be set to a given level datum). Right angles also can be produced quickly and precisely with auxiliary equipment components. In the assembly, maintenance, and calibration of industrial equipment, or alignment of precision systems, up to four basic alignment elements are used.

- straightness
- flatness
- squareness
- plumb

A number of techniques have been developed to make these measurements and in some cases it is no longer necessary to interpret readings or to make constant adjustments and calculations to produce accurate results. For example, in laser alignment applications, direct precision measurements are made rapidly and consistently based on existing technology.

*c. Straightness.* In aligning several points, a tight wire is often used as a reference line. This technique has some drawbacks and can introduce inaccuracy. Wire has weight, which causes it to sag, and over long distances this sag can become considerable. Wire vibrates, can bend or kink, and when stretched in the area to be measured, and equipment cannot be moved around for fear of disturbing the wire reference line. Even a gentle breeze can cause the wire to move a considerable amount because of the relatively large aerodynamic drag on a thin wire. In laser alignment, the Line of Sight (LOS) is established by a laser beam instead of a tight wire. The laser LOS reference has no weight, cannot sag, kink, or be disturbed, nor is it a safety hazard. It constitutes a precise reference for determining straightness to within thousandths of an inch. Straightness applications are employed for establishing an alignment survey reference line.

*d. Flatness.* A shop level and a straightedge are traditionally employed to determine flatness. The shop level must be moved from part to part over large horizontal areas to measure the degree of flatness of each individual surface upon which the level is placed. Flatness over a considerable area must be assured in the erection of large machinery, surface tables and large machine tools. Conventional bubble levels and "laser levels," offer a way to produce a level datum over a wide area. Laser technology has overcome the many disadvantages of bubble levels and assures levelness to within a few thousandths of an inch over hundreds of feet. This high degree of levelness is accomplished by horizontally sweeping the laser beam manually or via a motor driven rotary stage. This revolving line of laser light becomes a horizontal "plane of sight," giving a precise horizontal reference datum, sometimes called a waterline.

*e. Squareness.* Perfect squareness implies that one plane forms a 90 degree angle with another intersecting plane. When a steel square is used to test for this condition, measurements rely upon the trueness of the steel square, which can vary from square to square with time. Steel squares have a definite limit in their physical dimensions and consequently the testing of very large surface becomes less accurate and slower. Laser alignment methods use a transparent penta prism in conjunction with a simple alignment laser. This optical element will split the beam from the laser into two parts; one beam passes through the prism undeviated, the other beam is reflected at a 90 deg angle. Other systems use three independently mounted lasers that are orthogonal to each other.

*f. Plumb.* A plumbline and pendulum are used to establish a single vertical reference line. As vertical distances increase, settling time, vibration, air currents, and other disturbances will have increased effects on the measurements. In the laser alignment method there are several ways to produce a plumb reference; it can be a plane or a line. To form a plumbline, an alignment laser with auto-collimating capability is used with a pool of almost any liquid. Autocollimation senses the angle of an external mirror by reflecting its beam back into the laser head. A position sensor, beamsplitter, and lens measures the angle of the reflected beam. When the laser is adjusted such that the internal sensor reads zero in both axes, then the laser is producing a plumb line. If the laser beam is emitted from a manual or motor driver rotary base whose rotary axis is level, then the swept plane of light is a vertical plane. Position detectors in this plane will give an indication of how far to one side or other they are with respect to the plane.

### 3-12. Laser Tooling Methods

*a. Straightness alignment.* Before lasers and electronic targets came into use, alignment consisted of sighting through two points, near and far, and deciding if an object placed in-between them was to the left or right, or up or down with respect to this LOS. The choice of the two reference points is still the most important selection process of a straightness survey. For example, if a heavy machine tool is being surveyed, the two reference points that determine the LOS should be located off of the machine. If for any reason the machine were to move or to deflect, then all measurements would be in error. The two reference points should be located close enough to be convenient to use and/or out of the way of other people working in the area. Transits and alignment telescopes were the first instruments used to make these types of measurements. The use of transits and telescopes require one person to interpret a reading scale placed on the object of interest; and usually a second person is holding the scale against the object. It is a two person job that takes time and much training to accomplish. This type of alignment measurement, commonly called straightness, is the most basic of all alignment applications.

*b. Alignment transfer.* A another common requirement is to establish a second LOS perpendicular or parallel to an original LOS. To establish a perpendicular LOS with lasers, a special prism is used called a penta prism or optical squares as they are often called. Prisms have the property that rotation around its axis does not deviate the reflected beam at all, and therefore it does not have to be critically mounted. Tooling bars are also used to establish a parallel LOS with respect to an existing LOS, especially if the distance is relatively short, for example a meter or less. These bars are made of steel and hold electronic targets at a precise distance from a center point. Using two bars from the original LOS establishes a parallel LOS. If the distance between the two LOS is large, then transfer can be done using the penta prism twice; the first time to turn the beam 90 degrees, followed by a certain distance, and concluded by turning the beam back 90 degrees. Care must be taken that two LOS are truly parallel which is usually confirmed from a level reference datum.

*c. Oriented alignment.* The next alignment application involves measuring the alignment error between two different LOS datums. A typical application is to determine the lateral offset and angular error between two shafts (alignment segments). The shafts essentially define the two LOS. The measurement is made by setting up a laser source parallel to one shaft. Targets are placed on the second

shaft and surveyed in. Then the shafts are rotated 180 degrees and surveyed in again. The measured survey difference is equal to twice the shaft offset. If the target is placed at two axial locations and measured for offset, then the difference in the offsets divided by twice the axial separation is the angular error in radians.

*d. Alignment plane.* A more sophisticated alignment application is to quickly sweep a laser beam to generate a plane of light. The advantage of this is that many targets can be aligned using one laser source. In simple straightness applications the target location is restricted to the active area of the position sensor. In swept plane alignment the targets are sensitive in only one dimension. A typical application to establish a level plane is to put three or more targets at the same (desired) waterline location and adjust the structure to the targets until all targets read the same. The targets for a swept plane alignment can be static, meaning they require the laser beam to be constantly directed in to them. Usually the laser beam is swept by hand by rotating a knob on the laser source. If the laser plane is moving at high speed, say once a second or faster, then the targets must capture and hold the position of the laser beam as the beam sweeps by. The problem becomes harder to accomplish at longer distance because the beam is on the detector for such short periods of time. Physical high and low spots can be discovered and measured by moving the targets around the surface.

### 3-13. Laser Alignment Technology

*a. General.* The first laser alignment systems appeared in 1961 shortly after the invention of the helium-neon (HeNe) laser. The HeNe laser was the first practical way to produce continuous wave (CW) light. The high degree of coherence and Gaussian intensity profile allowed it to be easily collimated, or formed into a beam that could propagate a long distance without much spreading. Usually the 1 mm diameter of the HeNe laser was expanded to 6 to 12 mm to provide for good collimation over a useful range. The physics of propagating laser beams dictate that the larger the initial diameter of the beam, the less it will spread. Position sensitive targets that can intercept the laser beam at various places along the path of the beam will provide a straightness measurement and a simple concept for an alignment system.

*b. Alignment targeting systems.* The first 2D position sensitive targets initially consisted of four square photodetectors grouped together in a 2 x 2 arrangement called a quadcell [Discol, 1978]. The laser beam position on the surface of this target was computed with analog signal processing. The most basic target alignment method simply detects when the beam exactly straddles the boundary between two photodetectors. This nulling system was very repeatable and it gave the same accuracy independent of the power of the laser beam. This method does not give meaningful data when the laser beam is displaced from its nulled position. Developments in targeting technology made since the 1970's are described in the following paragraphs.

(1) Position sensor photocells. The first position sensitive targets appeared in the early 1970's. These used the difference in the outputs of two photocells, opposite each other, to measure displacement. This method was accurate to about 1/8 of a laser beam diameter. Measurement beyond this distance caused the difference (displacement) signal to decrease in value, finally reaching a terminal value when the laser beam was completely on only one photocell. In fact, with a quad-cell (or bi-cell target for 1D applications) it is never possible to measure any farther than 1/2 of a beam diameter from side to side. Another major drawback in this method is that the measurement is proportional to laser power. Variations in power received on the detector due to atmospheric attenuation, laser warm-up, power supply or temperature, would require manual adjustment of signal gain. An interim solution was to frequently check the displacement value a given target was producing with a field checking fixture. This item was nothing more than a cylinder that slipped over the front of a target containing a 1/4 inch thick glass window tipped at a small but precise angle. This fixture produced a known lateral displacement of the

laser beam at the surface of the target. If the measurement was too large or small, then a pot was adjusted to return the measurement to its correct value.

(2) Improved signal processing. The next development in target technology occurred in the mid-1970's to improve the signal processing and produce a displacement signal that was independent of laser beam power. This was done using an integrated circuit called an analog divider. Analog dividers were formerly large, rack mounted instruments that had been drastically reduced in size and cost to a single integrated circuit with the advent of microelectronics. The measurement signal was computed by dividing the difference of the two photocell outputs by their sum. Since both the difference signal and the sum signal are proportional to laser power, dividing one by the other results in a ratiometric signal that does not depend on incident laser power, and so it truly measured laser beam position on the target. Significant disadvantages remained that were a nonlinear measurement, a linear measurement range restricted to about 1/8 of a beam diameter, and sensitivity to ambient light. Ambient light could be occluded by the use of tubes placed over the ends of targets or by using interference filters which rejected any light not of the laser's wavelength. But these filters are expensive and tubes are cumbersome. The effect of ambient light or shadows cast on the surface of the detector could be rejected if the laser was amplitude modulated. However, modulating a HeNe laser is particularly difficult because of the 1000 volts required to keep the plasma tube excited. Practically, only a 10% modulation depth is achievable on a HeNe laser. This essentially cuts down on the useful signal level by a factor of 10 because the static or DC level of the laser is rejected by the processing circuitry.

(3) Measurement range linearity. The advent of lateral effect photodiode (LEP), or Walmark photodiode in the late 1970's allowed for larger and more linear measurement ranges. The LEP is a planar piece of doped silicon that produces a signal that is proportional to the intensity and the position of the "center of intensity" of the light falling on it. Unlike a quadcell, the LEP does not range saturate when the light spot has moved more than 1/2 beam diameter. The LEP produces a monotonically increasing signal as the light spot moves across its surface. The LEP does not distinguish structure, that is, it is not an imaging sensor. It will produce the same signal if a small diameter laser beam of a given power strikes it, as well as the same power spread out in a large diameter. This is usually not a problem. One advantage of the LEP is that it is very fast compared to photocells; some have an upper frequency limit of a megahertz. The typical LEP was termed a tetralateral type as it had 4 electrodes and a ground return path. These LEP types still exhibited some linearity errors at measuring ranges farther than 25% of its active diameter. Today there are duo-lateral types with shaped electrodes on the planar surface that give a very linear signal. Some targets now use CCD (charge coupled array) detectors, as for example, those typically used in video cameras. CCD targets are much more expensive and slower than LEP types. They have one big advantage; since they can sense structured light they can determine the centroid of a beam even in the presence of noise or a non-circular beam. They do this by using digital signal processing (DSP) techniques. Therefore, these type of targets are more expensive because the signal processing required is actually image processing, which is computationally time intensive. As the price of CCDs and DSPs come down and their speed goes up, more and more targets will use CCD array as their optical position sensors.

(4) Digital signal processing. Microprocessors appeared in the early 1980's and allowed greater flexibility and processing of signals. Now a system could be almost entirely digital in nature. This allowed them to be connected to networks and send their data over great distances. An LEP could be corrected for its errors by calibrating it during manufacture and storing the errors in a software look up table or by using curve fitting routines. When a measurement is made a curve fitting routine adjusts the raw data from the LEP into a very accurate displacement signal. One huge advantage with this technique is that it allows all targets to be metrologically identical independent of the particular LEP used. The lookup table can be stored in a memory chip right in the target, next to the LEP. Usually this is done with a non-volatile memory component such as an electrically erasable read-only memory. Finally, the

duolateral LEP appeared in the late 1980's that essentially provided better than 0.1% linearity across the whole detector surface.

*c. Laser sources.* As mentioned previously, the first sources of laser light were HeNe lasers. At the beginning they had a very short life, usually less than 1000 hours. Eventually their lifetime was increased by perfecting the glass-metal seals of the plasma tube. One bad characteristic of He-Ne is their efficiency which is less than 0.1%. Virtually all of the 5W or so of power required for a 5 milliwatt laser appears as heat. Recent developments in laser source technology are described in the following paragraphs.

(1) Pointing error. A critical design characteristic for an alignment system is the pointing error of the laser. For gas lasers such as the HeNe, the direction that the laser points at startup is not going to be the same as what it points to after 1 hour due to drift. Typical drift rates are 0.1 to 1 milliradian per hour and maximum drift can be as large as 10 milliradians. This amount of pointing error would cause a 1 inch shift at 8 feet. Plasma tube type lasers such as the HeNe are notoriously bad for pointing stability. Even after they have warmed up, a gentle breeze across its case will cause the laser beam to steer in a different direction. This type of error always causes errors in measurement unless the operator can make the measurements in less time than the drift occurs, or re-aims the laser at a known reference point frequently.

(2) Laser diodes. In the early 1990's the first visible laser diodes were introduced for use as a collimated source. They are small, low cost, require very little power, have efficiencies of 5 to 10% and vanishingly small drift rates. Optically, however, they are inferior to gas lasers. The light from a typical laser diode is emitted from a small rectangular aperture about 1 x 3 microns in size. Because of this small aperture the light diffracts, or spreads strongly with distance; it also has two different spreading angles because the aperture is rectangular. If a good quality lens is placed such that its back focal length coincides with the laser diode emitting surface, the beam produced will be elliptical in cross section and suffer from astigmatism. The astigmatism is a consequence of the aperture and results in a beam that always has two waists instead of one. Much effort is required to transform the light from a laser diode into a high quality collimated beam appropriate for use in precision alignment systems. Moreover, the LEP detector works best with a beam of circular cross section and that has one waist. Therefore, three approaches are used to improve the quality of a collimated beam from a laser diode; internal and external cylindrical optics; external prism optics; and fiber coupling.

(3) Cylindrical lens optics. A cylindrical lens is used to make the diffracted light emerging from the laser diode to have the same diverging angles in both axes. It is now possible to buy a laser diode with this lens inside the typical 9 mm diameter by 5 mm long laser diode case. Unless this lens is chosen carefully there still can be significant astigmatism in the optical system.

(4) External prism optics. External prism pairs can be used to circularize the laser beam, but it does not solve the astigmatism problem.

(5) Fiber coupling optics. The best way found to date that lets a laser diode have most of the same properties as a HeNe laser is to couple the light from a laser diode into a single mode optical fiber. This is usually done inside a small package that integrates the laser diode with a pair of aspheric lenses that efficiently couples the light into the fiber. The light emerging from the other end of the fiber is of uniform cross section, has no astigmatism, and has a well defined diffraction angle. The fiber end is then placed at the back focal length of a lens. The collimated beam produced is nearly the same as that produced by a HeNe laser. By choosing different focal length lenses the laser beam can be of any diameter desired. The drift rates of these laser sources are caused not by the laser, but by the package in which it is enclosed. If a steel case is used, maximum drift can be as low as a few arc-seconds. The light from a laser diode is polarized in one plane. The fiber coupling method does introduce a random

polarization to the beam after it has traveled through the fiber. Randomly polarized light is usually not a problem for an alignment target consisting of a lateral effect photodiode.

*d. Mechanical tooling.* In all laser measurement applications a question always arises as to how to mount the targets and laser sources. Usually commercial equipment vendors will supply their own proprietary mounting hardware. There is only one non-proprietary optical tooling standard for precise positioning of targets and lasers. It is called the National Aerospace Standard (NAS) and is based on all components fitting into precision 2.25 inch diameter bores. The NAS mechanical interface is used for locating and mounting of all optical tooling instruments. This universal mounting system consists of a tooling sphere and a 3 point cup mount. The tooling sphere is a truncated 3.5 inch diameter steel sphere. These sphere are 2 inches thick and have a 2.25 inch diameter bore machined precisely though the center of the sphere. The optical target, or laser source is inserted into the bore of the sphere and then the sphere is mounted onto a three point mount and clamped. The targets are designed so the optical sensors sit exactly at the center of the sphere, and if the target is tipped slightly, then the reading doesn't change.

### 3-14. Laser Alignment Techniques

*a. General.* Different techniques for conducting laser alignment surveys are presented in this section. These are related to methods for conducting surveys using single target alignments, two target alignments, and laser scanning systems.

*b. Single target laser alignment.* The main disadvantage of early laser alignment systems is that they only employed a single target. The target placed at a reference station establishes one end of the LOS and the center of the laser beam is the other end of the LOS. For single target surveys, the laser source is first carefully aimed at the center of the target, then the operator moves the target from its reference position to use it at intermediate locations. There are two problems with single target laser alignment:

- (1) the operator is unaware of any movement of the laser beam; and
- (2) alignment errors are introduced unless the reference laser position is frequently checked.

The only way to check for beam movement is to stop alignment operations, remove the target from its working location and move it to the reference station position. The position of the laser beam on the target at the reference station is then checked, and the laser beam re-aimed if necessary. This method is only useful for detecting slow variations in laser beam position at the reference station, for example, caused by thermal disturbances in the structure being aligned, or in long-term (e.g., geologic) instabilities at the laser source location. High frequency disturbances such as vibration can not be corrected at all.

*c. Two target laser alignment.* If two targets are used, then the measurement becomes more accurate because of the addition of a reference target situated at the far end of the LOS that constantly monitors beam position. The intermediate target used by the operator must allow for passage of light through to reach the reference target. The intermediate target is called the alignment or transparent target. In this approach, two different pointing compensation methods are used with transparent targets, namely, passive and active systems.

(1) Passive pointing compensation. If the two dimensional coordinates of the laser beam on the reference and alignment target are measured simultaneously, then the position of the alignment target with respect to a line between the laser and the reference target can be measured independent of any pointing error of the laser. The laser beam need not even be precisely aimed onto the center of the reference target. Instead, the coordinates of the laser beam at both targets are used to compensate for any laser beam movement. When the position of the laser beam is sampled rapidly, the system compensates

for thermal pointing errors, initial alignment errors, and vibration errors. In Figure 3-9 the line between the center of the reference target and the center of the laser beam source defines the LOS, not the laser beam. The laser beam is shown directed upward representing a laser pointing error. The transparent alignment target is shown centered with respect to the LOS. The pointing or wedge error as measured at the reference target is (h), because of similar triangles the pointing error is  $h'$  or  $(d/D) \cdot (h)$  at the alignment target. Subtracting this error from the measured beam position at the alignment target results in a compensated ( $C_{XY}$ ) alignment measurement, or true position of the target in both the x and y axes:

$$C_{XY} = h'_{XY} - (d/D)(h_{XY}) \quad (\text{Eq 3-6})$$

The LOS is defined by two points: one point being the center of the laser case and the other being the center of the reference target. The constants d and D are measured in the field or have been previously entered into the computer. Absolute target distances are not required, only the ratiometric distance, d/D. In some applications absolute distances are known and entered into the computer interface. In other applications ratiometric distances are more convenient to use. This technique is particularly useful at long laser-to-target distances, as angular errors at the laser create large position errors at the targets. Another advantage of passive pointing compensation is that the operator does not have to precisely aim the laser to dead center on the target. This allows operators to quickly set up the system. Because of how this technique uses geometric principles, it is called similar triangle compensation or passive pointing compensation.

(2) Active pointing compensation. Perhaps the most recent method to compensate for errors in steering the laser beam, due to thermal, mechanical or atmospheric effects, is to actively steer the laser. This technique uses all of the same components as passive pointing compensation, except the laser is now fitted with internal or external pitch and yaw pointing actuators. The reference target sends its error signals back to the laser where beam steering occurs to null out the error. The system acts as an optical servo mechanism. One advantage of this method over that of the similar triangle method is that absolute or ratiometric distances are not needed. Since the laser is always on the reference target center, no mathematical compensation needs to be applied. Any transparent target placed in the beam at any distance from the laser simply determines beam position. In this method the laser beam is the LOS.

(3) Scanning systems. Scanning systems can be a simple single-axis laser system that is manually rotated, or have 3 axes with each axis motor driven. The simplest laser sources for these types of system are small boxes with leveling feet and bubble vials. The user must set up the source to a level condition before it can be used. For three axis systems, once a level has been established, the other two axes will sweep out vertical planes of light that are perpendicular to each other. The most sophisticated scanning sources sweep out the beam automatically via a motor and they also contain internal level sensors. Some even control the degree of "levelness" by servo correcting the source if it moves off of level. A simpler method uses a pendulum on which a lightweight laser diode source is attached. The targets for these type of sources are always one dimensional. For a manually rotated source the electronics are similar to 2D targets; the user must aim the beam by hand into the target's window. For dynamic scanning the targets use very fast detectors as the beam sweeps by in only a few microseconds for a target located at some distance and a scan rate of 60 RPMs or higher. Sometimes LEPs are used as the sensor. For very high speed systems a bi-cell sensor is used. The sensor is rectangular and oriented in its long direction. A typical size would be 30 mm tall by 5 mm wide. But this sensor is split along its diagonal into two triangular shaped photodetectors. The two triangular shaped parts of the bi-cell are each connected to a timing circuit. When the time the laser beam spends on each segment is equal, the beam is exactly in the middle of the bi-cell. Deviations up or down produce a difference in timing that is exactly proportional to distance. The main advantage of scanning systems is that many targets can be placed in the scan zone.

It has a 360 degree scanning window and is designed mainly for leveling applications with accuracy the same as simple laser alignment.

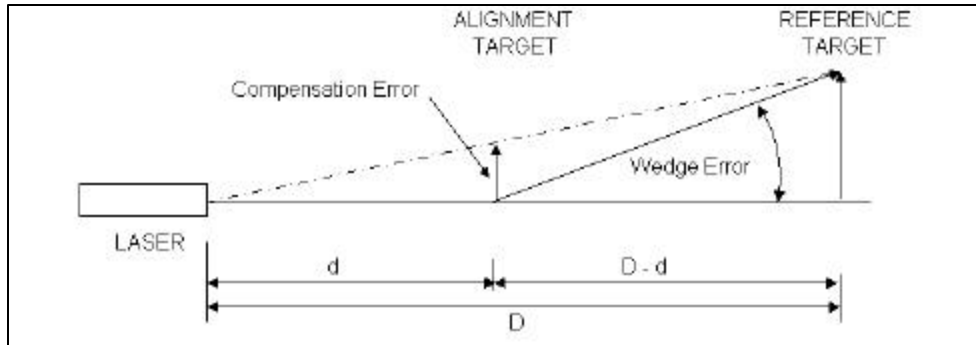


Figure 3-9. Geometry of a two target laser alignment.

### 3-15. Laser Alignment Error Sources

*a. General.* Any laser alignment system has associated measurement errors. Even if active and passive pointing compensation is not employed, any transparent target must not produce steering or deviation of the laser beam as it passes through it. The system's accuracy depends on the laser beam traveling in a straight line from the laser, through (possibly several) transparent target(s) and finally to the reference target. The transparent target will usually have: some sort of a beam splitter; and have windows on each end of it. Each window and the beam splitter possess a small amount of wedge error that acts to mis-steer the beam. Although the wedge error of these components is usually small (tens of arc-seconds), at long distances the displacement error can become large. There are two types of errors which can be injected into the error compensation equation; that due to steering of the laser beam by the transparent target (wedge angle error), and that due to the target being slightly tipped (deviation error).

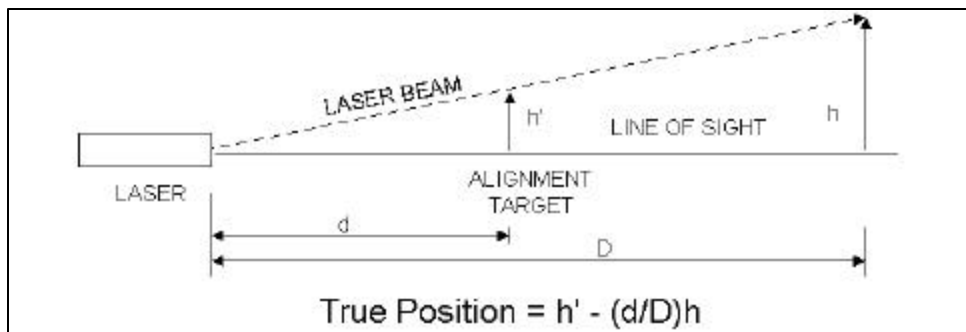


Figure 3-10. Two target alignment showing wedge error.

*b. Wedge angle error.* Rotation adjustment of the wedge prisms on the transparent target allows for the refractive error to be adjusted to less than one arc-second. Figure 3-10 shows a two target system with the laser beam initially centered on the alignment target. The alignment target is shown with a wedge error of  $\delta$  and it steers the incident laser beam away from the LOS. The laser beam strikes the surface of the reference target at a distance of  $(D-d)\delta$  from its center. The compensation equation then produces an error,  $\epsilon$ , of magnitude:

$$\epsilon = (d/D) \delta (D-d) \quad (\text{Eq 3-7})$$

due to the wedge error  $\delta$  of the transparent alignment target. Inspection shows this error is zero when the alignment target is situated at a distance of 0 or D from the laser source. If the alignment target was situated next to the reference target ( $d = D$ ), then it would impart no significant steering error at the reference target. If it were located next to the laser ( $d = 0$ ), then the wedge error as seen at the alignment target is also zero. The error is greatest when the alignment target is located halfway between the laser source and the reference target. Table 3-1 below shows how transparent target wedge error affects system alignment accuracy as a function of laser-to-reference target distance (D). The table assumes the alignment target is situated at D/2, or at one-half of the laser-to-reference target distance.

**Table 3-1. Compensation Error Due to Residual Target Wedge Angle**

Wedge( $\delta$ )	50-ft	100-ft	300-ft
10 arcsec	0.0075 in.	0.015 in.	0.045 in.
1 arcsec	0.00075 in.	0.0015 in.	0.0045 in.
.5 arcsec	0.00037 in.	0.00075 in.	0.0022 in.

*c. Target deviation.* Another error source is due to tipping of the target, causing a deviation of the laser beam as it passes through the windows and/or beam splitter. Deviation errors do not grow with distance as do pointing errors. Table 3-2 below indicates the magnitude of deviation error due to target tipping in yaw or pitch for a total glass thickness of 8 mm. This thickness represents the thickness of the windows and beamsplitter in a transparent target.

**Table 3-2. Deviation Error Due to Target Tipping**

Tipping Angle	Error
1 degree	0.0018 in.
3 degrees	0.0054 in.

*d. Target accuracy.* Most modern position sensitive targets used in alignment contain dedicated microprocessors. They can communicate their data over a bus or through the air with optical data links. The electronics of each target usually consist of a microprocessor; non-volatile, electrically erasable programmable memory (EEPROM); analog-to-digital converters; filters and serial communication drivers. Targets come in a variety of different sensing areas and virtually all use lateral photodiode detectors to sense laser beam position. Quad-cells are almost never used due to their small sensing range. Since each lateral photodiode detector has slightly different linearity, manufacturers now calibrate each target on a precision motion platform. Stored within each target are corrections for its detector. The result is that all targets are metrologically identical. A good rule of thumb for target accuracy is 1 part in 500 of a target's sensor diameter. For example, a target with a LEP sensor diameter of 1 inch would have a position accuracy of  $\pm 0.002$  inches.

### 3-16. Laser Beam Propagation

*a. General.* A propagating laser beam does not remain parallel as is frequently assumed. Even with "perfect" projection optics the laser beam obeys the laws of physics; the dominant law here is diffraction. All laser beams follow a prescribed propagation characteristic that depends on two conditions:

- (1) how the beam was launched, and
- (2) what type of disturbances it encounters along its path.

The first is greatly controllable; the second is usually not.

*b. Laser beam launch conditions.* The only two parameters which govern how a laser beam behaves after it is launched are: initial diameter and wavelength [Yoder, 1986]. For a given wavelength, the larger the initial diameter the less the beam will spread with distance. For a given distance, a laser beam with a long wavelength will grow in diameter faster than a laser beam of a shorter wavelength. These propagation characteristics are embedded in the exact equation below which is a result of electromagnetic theory.

$$\omega_z = \omega_0 \cdot \sqrt{1 + (\lambda z / \pi \omega_0^2)^2} \quad (\text{Eq 3-8})$$

where  $\omega_0$  is the initial laser beam radius;  $\omega_z$  is the laser beam radius at a distance  $z$  from the source, and  $\lambda$  is the wavelength of light. At long distances the equation simplifies to:

$$\omega_z = (\lambda z / \pi \omega_0) \quad (\text{Eq 3-9})$$

It can be seen how two quantities govern beam spread; wavelength and initial diameter. A laser whose beam is approximately parallel over a reasonable distance is called a collimated beam. Approximately is an appropriate term, because any propagating laser beam has associated with it a waist, or the place along the beam path where it has the smallest diameter. Sometimes this waist is located some distance from the laser source. At other times the beam waist is at the laser source. The beam waist is chosen to be located at a certain point, and to possess a particular diameter, depending on desired beam propagation characteristics. There is a distance over which the laser beam remains essentially parallel which is called its depth of field. The depth of field of a propagating laser beam is defined as the distance over which the laser beam does not grow by more than  $\sqrt{2}$  of its initial diameter or its waist diameter. Table 3-3 below illustrates the relationship between initial beam diameter and depth of field. The beam waist is located exactly in the middle of the depth of field. The diameter of the beam waist is  $1/\sqrt{2}$  or 0.707 of the initial beam diameter. The chart assumes a wavelength ( $\lambda$ ) of 635 nanometers, which is the wavelength of the visible diode lasers used in laser tooling. The  $z$ -range over which this conditions holds for an initial beam diameter of  $d_i$  is:

$$Z_R = (\pi d_i^2) / (4\lambda) \quad (\text{Eq 3-10})$$

**Table 3-3. Depth of Field for Different Laser Beam Diameters**

Initial Diameter	Depth of Field (m)	Depth of Field (feet)
1 mm	1.2	4
4 mm	20	64
10 mm	123	403
20 mm	492	1614
25 mm	769	2522

*c. Atmospheric conditions.* A beam of light propagating in a vacuum obeys the laws of diffraction and is not affected by any other source. The index of refraction for a vacuum is defined as exactly equal to one (1). However, in an atmosphere the beam will behave differently. The index of refraction of air being slightly larger (than one) causes changes in the propagation characteristics of a light beam. Two dominant effects on the beam are to make it move or quiver, and another that is commonly called "scintillation;" which means the intensity of the light beam varies as a function of time.

(1) Refraction. Much work has been conducted on the effects of atmospheric turbulence on propagating light. The index of refraction of air being different along the path length causes these two effects. The index of refraction of air is strongly affected by temperature, and to a lesser extent pressure and water vapor pressure (humidity). An expression for the index of refraction ( $n$ ) due to temperature ( $T$ ), pressure ( $P$ ), and humidity ( $H$ ) is given by:

$$n = 1 + 10^{-6} (79/T) [P + (4800H/T)] \quad (\text{Eq 3-11})$$

where  $T$  is in degrees Kelvin ( $K^\circ = C^\circ + 273$ ), and where  $P$  and  $H$  are in millibars. It can be appreciated that it is a weak effect by the  $10^{-6}$  factor in front of the second term. For most applications the expression is simplified by keeping pressure at a normal 1013 millibars and ignoring humidity.

(2) Scintillation. Perhaps the best known treatise on how the atmosphere affects light and sound propagation was by Tatarskii (1959). He identified how wind velocity affects scintillation and what the power spectral density was of the scintillations. He also determined how random side-to-side motions are scaled with distance and how propagation was affected by different atmospheric conditions. Measurements were made for all regions of the atmosphere, from the ground, through the troposphere, and into the stratosphere. Most of the measurements involved the frequency of the scintillations and not the positional shift of the light beam. Precision optical displacement devices and even lasers were not available when most measurements were made in the mid-1940's and early 1950's. Perhaps the most important contribution made was the introduction of atmospheric structure constants. These parameters provided information on how turbulent the atmosphere was including correlation distance as one of these important constants.

(3) Correlation distance. This is lateral distance from a propagating beam of light under which the scintillation and turbulence would be completely different for a neighboring beam. For a quiet atmosphere where there is gentle and thorough mixing of the air layers, the correlation distance  $\rho$  is equal to [Smith, 1993]:

$$\rho = \sqrt{\lambda L} \quad (\text{Eq 3-12})$$

where  $\lambda$  is the wavelength of the light and  $L$  is the distance from the source. The correlation distance is important because it affects the choice of beam diameter. For example, if a range of  $L = 123$  meters is used (from Table 3-3, depth of field for a 10 mm diameter beam), then for a  $\lambda$  of 0.635 microns ( $0.635 \times 10^{-6}$  m) the correlation distance is 8 mm. What this means is that since the laser beam diameter required for good collimation is approximately the same size as the correlation distance, the beam will undergo a slight amount of fading. The fading is due to one side of the beam interfering with the other side, after traveling a long distance, and so experiencing a different atmosphere. If the beam were less than the correlation distance this effect would not happen. Indeed, experience has shown in the field that on "long shots" if one holds a piece of paper up to beam at a long distance from the source, the spot on the paper will change shape quickly. It will be circular one moment and highly elliptical the next. A non-circular beam will cause errors in laser beam position measurement, because LEP targets measure the centroid of the laser beam.

*d. Remedies to atmosphere effects.* There are a few ways to remedy these atmospheric error effects. One is to make certain that there is no cross-beam wind component. This can be achieved with tubes physically enclosing the beam path. Another method is to blow air down the laser beam path with fans. The idea is to eliminate cross beam wind components with a down beam velocity component. If air conditioning is used in the building, it should be turned off as the extremely cold air mixing with hot air gives rise to the worst beam deflections possible. Some other remedies are to change either the range or the wavelength used. For example, decreasing the alignment range to 50 meters would decrease the correlation distance to 5.6 mm. Conversely, if a blue wavelength laser were used with a  $\lambda$  of 0.42 microns the correlation distance would be about 6.4 mm at 123 meters. The basic consideration in long distance shots is to have the beam diameter as small as or smaller than the correlation distance. The highest expected frequency of scintillations is:

$$f(\mathbf{v}) = \mathbf{v}/(\text{sqrt}(\lambda L)) \quad (\text{Eq 3-13})$$

where  $\mathbf{v}$  the cross-beam velocity component of the wind. It should be noted that  $L$  in this equation and the above one can not take on any value--i.e., the range  $L$  must be located in the far field of the source. Usually, this distance is on the order of 10 meters. For the same situation as above with  $L = 123$  meters,  $\lambda$  of 0.635 microns, and a 1 meter/sec velocity, the maximum frequency of beam scintillation is 113 Hz. As in any data acquisition system, if one samples laser beam position at the target at least twice this frequency, then aliasing errors will not occur.

### 3-17. Laser Alignment Equipment

*a. Commercial systems.* This section describes some laser alignment equipment, from ON-TRAK Photonics, Inc. (shown in Figure 3-11), and AGL Construction Lasers and Machine Control Systems. Table 3-4 contains a partial list of manufacturers of laser alignment and scanning systems with tabulated measurement ranges, target capture areas, accuracies, and product data such as whether the vendor can design scanning, alignment, and custom systems. This list is not meant to be all inclusive.

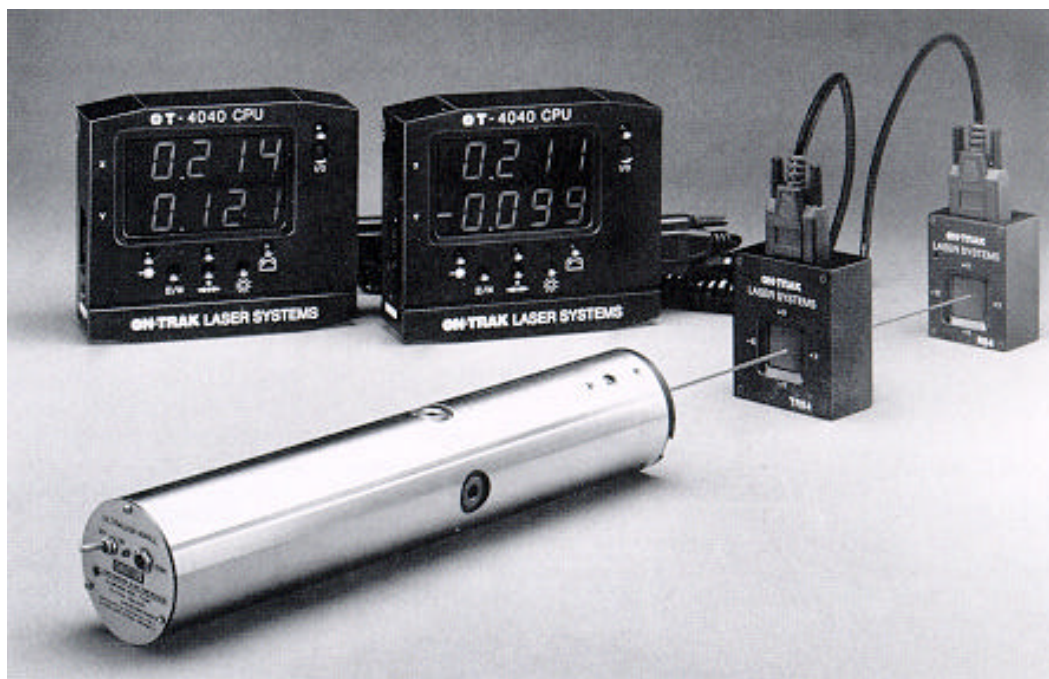


Figure 3-11. Laser alignment system from ON-TRAK Photonics, Inc.

Table 3-4. Laser system manufacturers and performance

Manufacture	Scan	Align	Custom	Range	Area in.	Accuracy in.
On-Trak	Y	Y	Y	2700 ft.	0.500	0.001
Hamar	Y	Y	Y	100 ft.	0.075	0.001
Pruf-Tecnik		Y		30 ft.	0.100	0.001
PinPoint		Y		30 ft.	0.250	0.010

*b. OT-6000 alignment laser system.* ON-TRAK Photonics, Inc. produces a commercial laser alignment system used for measuring 2D spatial deflections from a laser reference line. Components of the system are:

- OT-6010 Transparent Laser Alignment Target
- OT-6020 Reference Laser Alignment Target
- OT-6000 DIM Digital Interface Module
- OT-6000LL Alignment Laser
- OT-6000 DT Data Terminal

Components are sold separately and must be configured and installed on-site by the user. The next four sections describe each major system component by product type.

(1) Transparent Laser Alignment Target OT-6010. Raw measurement data is gathered by an alignment target sensor. The OT-6010 transparent laser alignment target consists of a 22.5 mm (0.885 DIA) diameter dual-axis sensor, with an active area set in a vertical 2D plane (X-Z position) orthogonal to the laser beam. A transparent sensor material, having >85% beam transmission, allows up to 6 targets to

be simultaneously aligned to a single laser reference. Each target piece is cased in a 3 inch square by 3.25 inch housing with a precision NAS Standard Tooling Sphere mount at its base. Power supply (standard 110V AC) and digital communications are made over the same cable using an RS-485 to RS-232 converter located in the OT-6000DIM Digital Interface Module. Windows Terminal, hyperterminal, or other standard communications software is required to operate the system. Target beam-center detection accuracy is 0.001 inch with programmable resolution set in increments of 0.0001 inch. Beam deviation through the target is <1 arc-second with a  $\pm 0.0005$  inch centering tolerance when mounted to the NAS tooling sphere.

(2) Reference Laser Alignment Target OT-6020. One non-transparent reference alignment target (OT-6020), with similar specifications to the OT-6010 targets, is used to terminate each installed series of transparent targets.

(3) Central Processing Unit (CPU) OT-4040. An interchangeable OT-4040 CPU system is required for each laser target. The CPU consists of a self-contained, battery operated laser signal processing unit that is networked to a host computer. The system uses an RS-232 serial communications port for data collection, target addressing, and self-calibration. The CPU Unit displays absolute X-Y position to the operator with a 0.001 inch resolution. Features include adjustable laser pulse averaging controls, programmable LED brightness, zero offset position adjustment, and target status by remote query accessed via the network and an RS-232 communications port.

(4) Alignment Laser OT-6000LL. A reference line for the survey alignment is established by a collimated source of laser light. The OT-6000LL alignment laser is a CDRH II class, 670 nm frequency solid state Diode laser that outputs a maximum 60 micro-Joules by 20 ms at 5 Hz, providing an operating range of 100 feet. It is enclosed in a 12-inch, hardened stainless steel casing with a Rockwell C64 hard chrome surface weighing 6.5 lbs (outside diameter is NAS standard 2.2498 inches, plus 0.000 inch and minus 0.0003 inch). The system is powered by an AC wall charger through internal NiCad batteries. Performance of the laser includes an alignment stability (drift) of less than 0.005 inches per hour, beam centering to within  $\pm 0.001$  inch relative to the mechanical center, parallelism within  $\pm 2$  arc seconds, and produces a beam diameter of 8 mm at 100 feet.

*c. AGL Total Control Laser (TCL).* AGL Construction Lasers and Machine Control Systems produces a commercial laser alignment and digital laser theodolite package used for tunneling, mining, and alignment control. Components of the system are:

- Laser transmitter
- Alignment Base Plate
- Digital laser theodolite

Components are sold separately and must be configured and installed on-site by the user. The next three sections describe each system component by product type.

(1) Laser transmitter. The AGL is a 1.9 mw He-Ne laser with a wavelength of 632.8 Nm. The system runs on 12 volt DC battery, is water resistant, with a length of 19.5 inches and a weight of 6.25 lbs. Sighting through the target set-ups is aided by a sighting telescope mounted to the top of the laser unit. The chart below (Table 3-5) gives manufacturer supplied beam diameter properties as a function of distance.

**Table 3-5. AGL laser system beam diameter and range.**

Range (meters)	Beam diameter (mm)
0	9.4
213	20
427	45
609	66

(2) Alignment base plate. The system operates with an assortment of mounting devices that attach to a tripod. A mounting plate is used to attach the instrument or targets to either a leveling base or a tilting base tribrach. In operation, the alignment control system is made by positioning three base plates in a row. The TCL system is mounted to the first plate, with in-line targets positioned on the other two plates. The targets are positioned at the desired line of control and the laser is adjusted so the beam passes through the hole on the targets. If the laser alignment is disturbed, the beam will become blocked indicating loss of control. The beam is used as a guide to mount additional targets down range as work progresses. The tilting base is used to control alignment attitude and the leveling base is used for straightness alignments with respect to the gravity vector.

(3) LDT50 digital laser theodolite. The LDT50 system combines a laser diode system and theodolite for alignment and orientation monitoring applications. The laser beam range in this system is over 1300 feet with two-stage power output and switching between a focused or a parallel beam mode. The theodolite has a dual-axis compensator for reducing leveling error.

### **3-18. Current Laser Alignment Surveys--Libby and Chief Joseph Dams, Seattle District**

The following sections are extracted from a USATEC 1999 report "Design and Evaluation of Geodetic Surveys for Deformation Monitoring at the US Army Engineer District, Seattle." This report contains technical guidance that may have Corps-wide application.

*a. General.* Laser alignment is a major tool in deformation monitoring surveys at both Libby and Chief Joseph dams (Seattle District). The technology currently used dates back to the late 1960's when laser was still a novelty in engineering applications. At that time, intensive research was conducted on the propagation of laser in turbulent atmosphere (Chrzanowski and Ahmed, 1971), and on development of time integrated and self-aligning laser centering targets (Chrzanowski, 1974). The equipment employed at the Libby and Chief Joseph dams is the simplest available at that time, consisting of a low power HeNe laser (with expected large thermal directional drift of its output), collimated with a 50 mm diameter collimator lens and a simple divided (Wheatstone bridge balancing) photodetector for sensing the center of energy of the laser beam. Since the detector does not perform a time integration of the scintillating laser spot, the alignment distance is limited by air turbulence to only about 250 m. The translation stage of the photodetector is also of a very old design equipped with a vernier readout whereas newer translation stages have micrometer or digital readout systems. Although the system can still give a resolution of better than 1 mm, it is cumbersome and labor intensive to use. The accuracy of the deflection measurements is designed to meet a 3 mm tolerance at the 95 percent confidence level. An upgraded laser system should have a beam-center detection precision in the range of  $\pm 1$ -2 mm at the 95% confidence level at its maximum operating range. At most a total uncertainty in deflection measurement should be no greater than 5 mm at the 95% confidence level.

*b. Alignment equipment.* The existing laser system has the following potential weaknesses:

- uses out-of-date technology;
- has a limited working range;
- requires excessive warm-up time;
- exhibits low reliability in beam centering at long range;
- lacks internal checks for station centering;
- lacks direct referencing to the alignment end-points;
- relies on manual reading and manual target alignment;
- experiences systematic drift in orientation during the survey;
- sensitive to atmospheric disturbances;
- affected by refraction errors in the measurements.

The current laser system uses a Spectra-Physics LT-3, Stabilite model 120T HeNe gas laser, (model 257 exciter), with a Model 336 Multiwavelength Collimating Lens, 450-650 nm, 50 mm CA, 200 mm EFL @ 587.6 nm. The target system is a custom fabricated light sensor (detector). The target housing consists of a 6-inch diameter cylinder with a single vertical splitter plate dividing the casing into two chambers (bi-cell). A cadmium sulfide photo resistive cell is mounted to each chamber and wired into a Wheatstone bridge circuit. In use the target sensor is known to be centered on the laser beam when the light intensity on the two photo resistive cells is equal as indicated by a null reading on a microamp meter. A translucent diffusion screen allows coarse beam centering to be performed by eye before fine measurement is attempted. The target sensor has both horizontal and vertical motion provided by a manually operated translation stage. Vertical travel range is 1.7 inches, horizontal travel is 4.0 inches. Target adjustments are made by threaded-rods (actuators) with the amount of travel referenced to horizontal and vertical vernier scales that are rigidly mounted to the target assembly.

*c. Alignment surveys.* Laser targets are forced centered into permanent floor or wall inserts next to each monolith joint. Offsets are recorded in the U/D stream direction between the target's zero position over the survey monuments, and the center of the laser beam. The operator translates the target onto the alignment by mechanical adjustment (actuators). Beam-position centering is made visually from meter readings and by moving the target until a null readout is obtained. The meter tends to display more erratic output as the target is moved to its maximum operating distance. Offset measurements are repeated three times on a vernier scale with 0.001-inch resolution. Reading errors are related mainly to system and operator bias, and to the operating resolution of the vernier scale. Data is recorded by hand on standard data cards that are printed on sheets of paper and then the data is transferred a PC text file.

*d. Gallery environment.* Each gallery has overhead lamps that are turned off during the survey. Flashlights are needed to navigate between target stations. Movement in the gallery is restricted to avoid creating air currents across the laser beam or near the targets. Disturbed atmospheric conditions may delay work for up to 10 minutes after walking near a target station. This slows down the measurements and moving equipment during the surveys. Localized air turbulence also influences beam collimation and laser accuracy/detection performance. It takes roughly 3-15 minutes to finish the readings at a single laser target station. Setup and warm-up times are approximately 30-40 minutes for each time the laser is moved to a new base segment station. Average survey completion times are on the order of 7-8 hours per gallery.

*e. Laser survey procedures.* Due to the length of the laser survey at the two dams (760 m at Libby Dam and 590 m at Chief Joseph Dam), the survey is broken into a number of segments. The adjacent segments are oriented relative to one another using common points observed in the overlap

region. This observation scenario is depicted in Figure 3-12. The offset measurements from the different laser segments are referred to the baseline between the two reference endpoints.

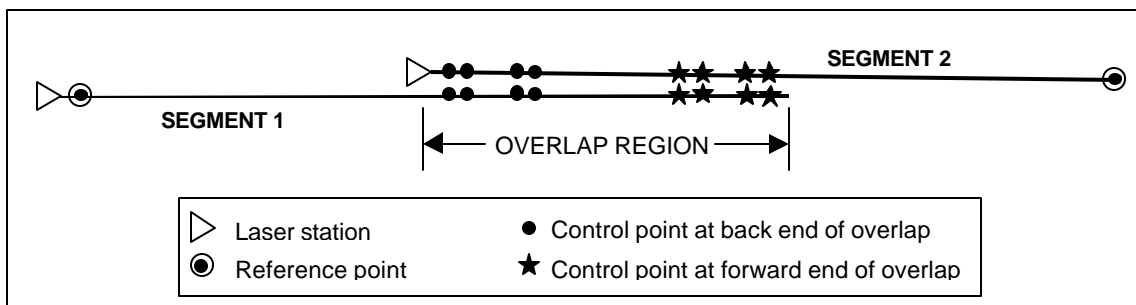


Figure 3-12. Connection of laser survey segments by observation of common points.

Deterioration of the laser beam image with distance from the transmitter has restricted the segment length to approximately 800 feet (240 m), with about 250-340 feet (75-100 m) of overlap between segments. This results in a five-segment survey at Libby Dam and a four-segment survey at Chief Joseph Dam.

*f. Laser surveys and refraction.* It has been observed that there is no way to quantitatively evaluate the problem of long-period distortions of the reference line due to refraction (which could be caused by horizontal temperature gradients), while the high-frequency, small-amplitude oscillations caused by atmospheric turbulence could be reduced by limiting the length of the traverse segments. High-frequency oscillations caused some difficulties for the observer in photo-electrically centering the target on the laser beam. Therefore, a survey procedure was developed which minimized the effects of atmospheric turbulence. At each survey segment, the survey would proceed from the end target (the one farthest away from the laser) to the closest target, so as to minimize the disturbance of air between the target and the laser. This procedure minimizes the effect of atmospheric turbulence on the readings, and allows the observer to collect a set of readings which have good internal precision (i.e., they are close to the same value). However, the effect of atmospheric turbulence is random, and thus will be averaged out if enough measurements are taken. The survey procedure does nothing to minimize the much more serious and difficult to detect problem of systematic refraction caused by horizontal temperature gradients. In fact, the effect of refraction is most pronounced when the measurements are collected in this way, because the air between laser and target is allowed to settle into thermal layers. The refraction problem can be reduced by disturbing or mixing the air between laser and target, which causes the refraction to be randomized. This would also affect the internal precision of the survey, but the overall accuracy would be improved. At both Chief Joseph and Libby dams, a horizontal temperature gradient could exist due to the fact that one wall of the gallery is closer to the pool while the other wall is closer to the outside air. Even if there is no gradient due to the two walls being different temperatures, it is quite possible that the walls themselves have a different temperature than the air in the gallery. For this reason, it is crucial to keep any optical lines of sight as far from the wall as possible. At Chief Joseph dam, the laser survey is run down the center of the gallery; this is the best possible place for the survey. At Libby Dam, however, the survey is performed very close to the wall (less than 30 cm).

*g. Alignment calculation procedures.* To determine the relative orientation of the laser segments, a procedure is used as illustrated in Figure 3-13. For each overlap region, a number of points are called 'forward' overlap points and others are called 'back' overlap points. Numerous trials are conducted to fit each possible pair of back and forward overlap points as 'control' points. In each trial, two control points are fixed to be coincident, and the average error of superposition is calculated for the rest of the overlap points. The combination of control points that yields the lowest average error of superposition is chosen to define the relative orientation of the adjacent segments. For the remaining overlap points, readings

from the forward segment (i.e., the segment where the overlap points are closer to the laser) are used in the offset calculation, and data from the back segment is discarded. When all of the segments have been processed for relative orientation, the whole line is constrained by setting the offsets of the two endpoints to zero.

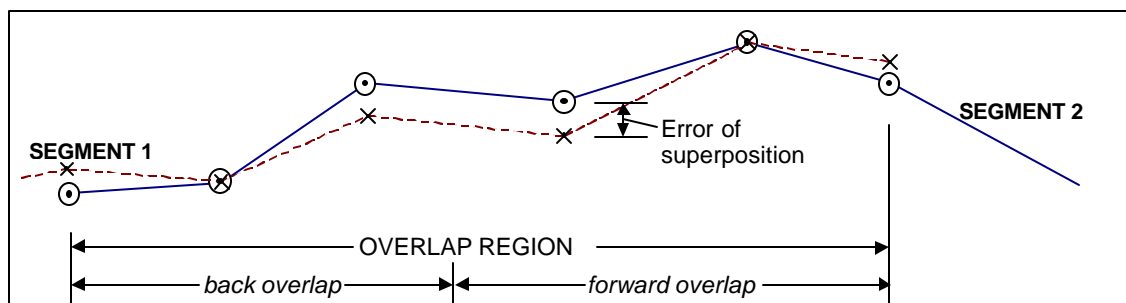


Figure 3-13. Segment orientation using two overlap points

Reducing the laser survey data by this method has several drawbacks:

- the overlap error at two points is artificially constrained to be zero, when it is known that these two points are observed with the same level of accuracy as the other overlap points.
- the method does not make optimal use of all available data. For example, data from the back segment of the overlap is discarded. Although the readings from the forward segment should be higher quality (due to the fact that the targets are closer to the laser in this segment), this is not enough to justify ignoring the only redundant observations made in the survey. A better procedure for orienting traverse segments is to use data from all of the overlap points, while incorporating the knowledge that measurements from the back segment have a higher standard deviation than those in the forward segment.
- there is no way to include any external information to yield a better estimate of the offsets. The laser alignment procedure has the same error propagation characteristics as a straight survey traverse; where the uncertainty perpendicular to the survey line increases with distance from the control points. If additional information is used to constrain the measured offsets along the traverse (e.g., from plumbline data), it would dramatically improve the precision of the results.
- statistical assessment of the offset solution is restricted to an analysis of the superposition error between individual segments. There is no calculation made of the estimated precision of the derived offset values based on the survey data.

In conclusion, it is recommended to use a more flexible and rigorous data reduction scheme, based on least squares methods, for processing the relative orientation of laser alignment segments.

*h. Laser survey accuracy evaluation.* From previous studies evaluating the precision of the laser system it was concluded that the probable error (i.e., at the 50% confidence level), of the derived offset is  $\pm 0.031$  inches (0.79 mm). This corresponds to a standard deviation of:

$$\sigma = \pm 0.042 \text{ inches (1.07 mm),}$$

meaning that the derived deflections would have a 95% confidence value of:

$$(1.07) (\sqrt{2}) (1.96) = 2.97 \text{ mm.}$$

This accuracy evaluation is a useful for understanding the internal precision of the alignment system. However, this evaluation does not account for the effect of systematic environmental influences and therefore does not indicate a real accuracy for the system. The data was reduced using four independent sets of survey control, but, all of the observations in each epoch were collected as part of the same observation campaign, and thus could have been affected by the same amount of systematic refraction. In order to get a valid accuracy assessment, there are two choices:

(1) Either, run the alignment survey several times under different atmospheric conditions and observe the spread in offset values. In this case, all of the surveys would have to be completed over a short period of time, so that movement of the structure does not affect the results.

(2) Compare the deflections obtained from the alignment surveys with collocated deflection data obtained from a different independent source.

The first option would require entirely new field observations. The second option, however, has been investigated using deflection data from plumbline readings at Libby Dam. A summary of this comparison indicates a standard deviation of 5.3 mm for the two sets of deflection values (i.e., differencing of pairs of data at the same epoch). A 95% confidence value of:

$$(5.3 \text{ mm}) (1.96) = 10.4 \text{ mm}$$

was obtained, and is a more realistic assessment of the accuracy of deflections from the laser alignment system. This result assumes that the Libby Dam plumbines have been properly installed and carefully observed. This accuracy level can also be used to approximate accuracies for the Chief Joseph surveys, bearing in mind that it is slightly shorter than Libby Dam, and that it is run down the center of the gallery rather than close to one wall.

### 3-19. Suspended and Inverted Plumblines

Suspended and inverted (floating) plumblines are among the most accurate, easy to maintain, and reliable instruments used in structural monitoring. The two inverted plumblines at Libby Dam Monoliths 6 and 46 monitor the stability of the end points of the laser alignment system. Plumbline readings since 1991 indicate that both monoliths are stable within  $\pm 0.25$  mm (0.01 inch) in the U/D direction and within  $\pm 1$  mm (0.04 inch) along the axis of the dam. Monoliths 23 and 35 contain both suspended and inverted plumblines. At each monolith, the suspended plumblines extend from the upper inspection gallery to the drainage and grouting gallery. The inverted plumblines extend from the drainage and grouting gallery to an anchor 10 m deep in the bedrock. Therefore, suspended and inverted readings at the drainage and grouting gallery can be combined to give the total displacement of the upper inspection gallery with respect to the bedrock. The combined readings at these two monoliths indicate very smooth cyclic deflections of the dam (particularly at Monolith 23 as shown in Figure 3-14). Movement is well correlated with the cyclic water load changes, with a maximum total range of deflections of about 18 mm (0.7 inch).

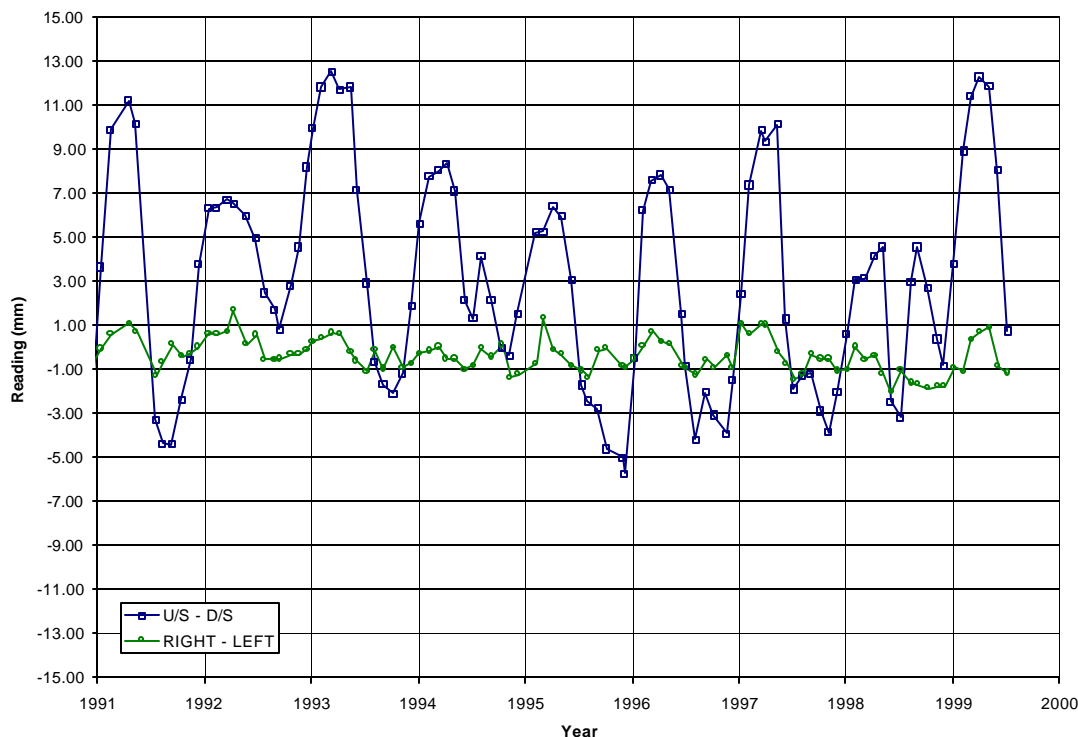


Figure 3-14. Combined readouts from suspended and inverted plumblines at Monolith 23, Libby Dam

### 3-20. Comparison of Alignment and Plumblines Systems

*a. General.* The results of the laser alignment have been treated as an independent survey without any attempt to correlate or integrate the results with indications of other instruments, particularly with the reliable plumblines measurements. As such, there was no control on the stability of the end points of the alignment line and no check on possible refraction effects. For example, at Libby Dam, alignment surveys have been carried out twice yearly, in May and in November. One should note that these two epochs of observations do not coincide with the maxima and minima of the dam deflections that occur in March and in September as indicated by the plumblines results. This is working against the principal rule stated earlier for monitoring the maximum expected deformation. Figures 3-15 and 3-16 show plots of the May and November laser survey displacements for the years 1991-1999, respectively. There are large changes in the displacements of individual monoliths from one year to another, reaching a maximum of 20 mm between 1997 and 1999. One cannot explain or correlate the results with water level or temperature changes. In order to interpret those deflections, the results at station 23R have been compared with plumblines readings interpolated to the time of the alignment surveys (see Table 3-6). Discrepancies between the two types of surveys far exceed the errors of plumblines readings that are estimated at 0.3 mm. The maximum discrepancy (31 October, 1994) reaches 8.1 mm, exceeding by 10 times the actual (plumblines survey) deflection of the dam in comparison with 1991 data. Using the discrepancies ( $\Delta$ ) from Table 3-6 as indicating true errors of the alignment survey, the error in the laser deflection survey is equal to 10.4 mm at 95% confidence level. This means, that when employing currently used procedures and calculation methods, the alignment surveys cannot detect displacements smaller than 10.4 mm.

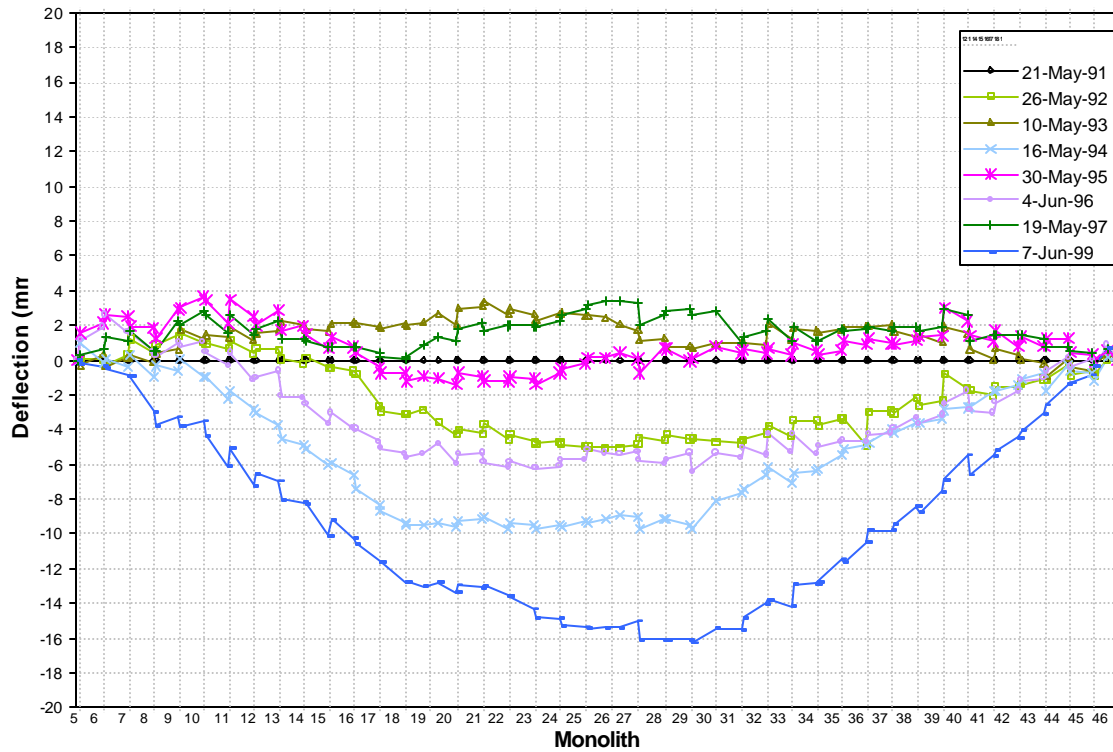


Figure 3-15. May results of Libby laser alignment (1991 base)

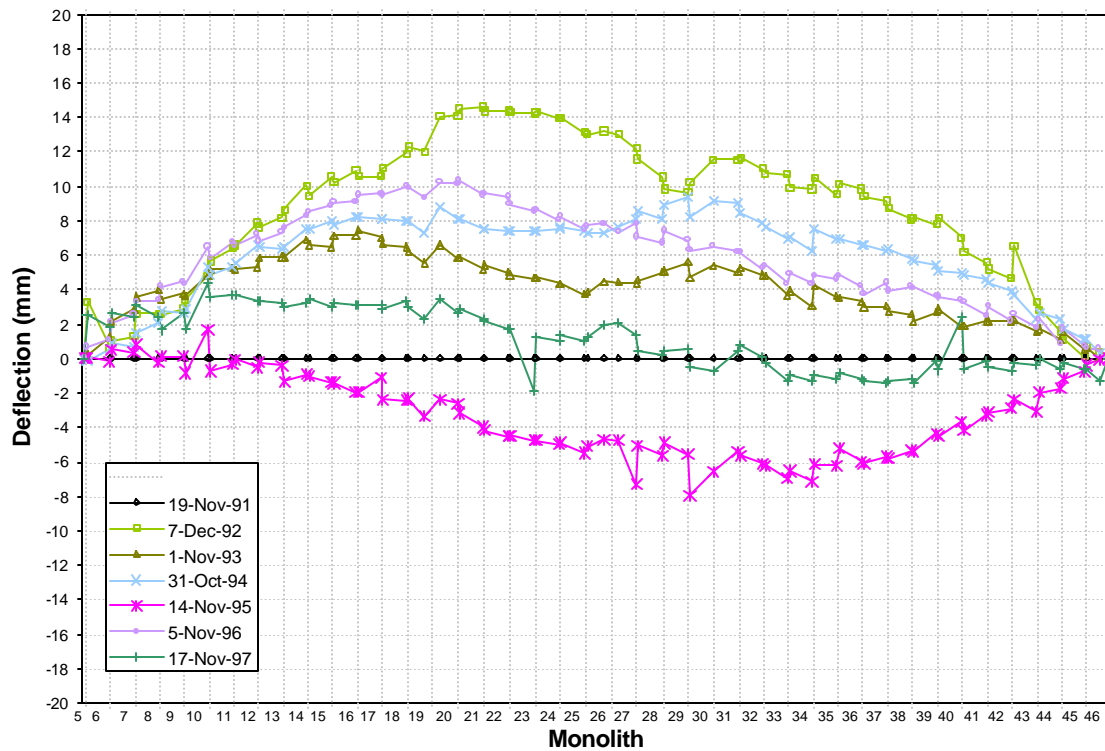


Figure 3-16. November results of laser alignment (1991 base)

**Table 3-6. Comparison of laser alignment with plumbline data**

Date	Plumb (mm)	Reduced (1991)	Laser (mm)	Diff $\Delta$ (mm)	Temp (°C)	$\Delta T/\Delta y$ (°C/m)
1991-05-21	6.9	0.0	0.0	0.0	13	
1992-05-26	5.8	-1.1	-4.6	-3.5	14	-0.05
1993-05-10	11.8	4.9	2.5	-2.4	11	-0.03
1994-05-16	5.2	-1.7	-9.4	-7.7	14	-0.11
1995-05-30	3.7	-3.2	-1.0	2.2	18	0.03
1996-06-04	3.5	-3.4	-6.1	-2.7	14	-0.04
1991-11-19	0.4	0.0	0.0	0.0	8	
1992-12-07	7.1	6.7	14.2	7.5	3	0.10
1993-11-01	-1.3	-1.7	4.6	6.3	12	0.09
1994-10-31	-0.3	-0.7	7.4	8.1	12	0.12
1995-11-14	-5.0	-5.4	-4.3	1.1	8	0.02

*b. Refraction effects.* The only reasonable explanation for the large error of the laser alignment surveys is the influence of atmospheric refraction. Using the values of ( $\Delta$ ) from Table 3-6, one can calculate the expected changes of gradients of temperature between the 1991 survey and subsequent epochs of observations. Those values are listed in the last column of Table 3-6, assuming a survey length of 760 m and atmospheric pressure of 1000 mb. The results vary between -0.11 deg C/m and +0.12 deg C/m. Considering the fact that the alignment surveys are performed within a few inches from the wall of the gallery, those values of gradient changes would be quite realistic.

### 3-21. Tiltmeter Observations

Tiltmeters require extremely careful and frequent calibration for temporal drift of the output, effect of temperature changes, and linearity of the conversion factor (Volts vs. angular units). Therefore, tiltmeters are among the least reliable instruments for permanent installations.

*a. Drift calibrations.* Accelerometer type tiltmeters should be calibrated for drift on a stable tilt-plate station, situated off of the structure, having a known or monitored reference tilt value. Drift error is modeled by solving for changes in tilt as a function of time at a reference tilt station immediately prior to and after each survey. Corrections are interpolated for each tilt measurement using observed time and drift rate from the model.

*b. Temperature corrections.* Before drift calibrations are computed, a thermal correction needs to be applied to account for changes in the shape of the accelerometer unit at ambient temperature. This is especially important when comparing tilt measurements made over different seasons of the year. A correction is based on a temperature coefficient ( $\Delta\alpha$ ) supplied by the manufacturer:

$$\Delta\alpha/\Delta T = \pm(0.03 \% \text{ reading} + 3 \text{ arc sec})/\text{deg F}$$

Actual temperature readings are made in the gallery during the tiltmeter surveys. Readings in areas exposed to sunlight should be taken in the early morning before thermal instabilities affect the shape of the structure. Final tilt angle values are converted to a length-to-distance ratio using a pre-selected baselength distance. Linear horizontal displacements are found using the elevation difference between each tilt plate and the bottom of the monolith (as a radius of rotation) assuming the monolith behaves as a rigid body. Higher resolution electrolytic tiltmeters are available that operate by a liquid bubble level

sensor (50 mm vial length). These instruments are permanently fixed to the structure and have a repeatability of  $\pm 3$  arc-seconds with automated reading systems.

### **3-22. Mandatory Requirements**

There are no mandatory requirements in this chapter.